VOLUME 83 NO. SA6
DECEMBER 1957
PART 1

JOURNAL of the

Sanitary Engineering Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

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This Journal is published bi-monthly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class mail privileges are authorized at Ann Arbor, Michigan.

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Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

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VORTEX FLOW THROUGH HORIZONTAL ORIFICES

J. C. Stevens,[1] M. ASCE, and Richard C. Kolf,[2] J.M. ASCE
(Proc. Paper 1461)

SYNOPSIS

A few years ago the senior author made practical use of vortex flow in connection with the diversion of sewage from combined sewers. By causing the sewage to approach a horizontal orifice tangentially, a vortex was set up.

Vortex flow through such orifices is given by the common formula

$$Q = CA\sqrt{2gH} \tag{21}$$

However, for a vigorous vortex from a storm head in the sewer, the coefficient is not the common 60% but may drop as low as 10% (Fig. 6) with the result that only a small portion of storm water can pass into the interceptor. At low stages however the entire sanitary flow is diverted. This device, therefore, becomes a very effective control without moving parts for diversion of sewage from combined sewers into interceptors. Facilities for research in Portland had to be confined to a few model experiments which, however, were amply sufficient to prove the efficacy of the vortex diversion. In order to go deeper into the subject, and particularly the theory involved, arrangements were made with the University of Wisconsin Hydraulic Laboratory to undertake a research project on this type of vortex flow. The junior author was assigned this project in partial fulfillment of the requirements for his doctorate.

The theory of vortex flow through horizontal orifices, the results of laboratory experiments thereon and some practical applications of the principles and data adduced, form the subject matter of this paper.

Note: Discussion open until May 1, 1958. Paper 1461 is part of the copyrighted Journal of the Sanitary Engineering Division of the American Society of Civil Engineers, Vol. 83, No. SA 6, December, 1957.

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Theory of Vortex Flow

Symbols

The following signs and symbols conform to the American Standard, Letter Symbols for Hydraulics ASA Z10.2 1942

A,	area of orifice
B,	diameter of boundary ring
C,	standard orifice coefficient of discharge
D,	diameter of orifice
$\underline{\underline{E}}$,	Euler number
$\underline{\underline{F}}$,	Froude number
F_c,	centrifugal force
F_i,	inertia force
g,	acceleration due to gravity
H,	static head on center of orifice
K,	vorticity factor
K_1, K_2	profile constants
p,	pressure per unit area
r,	radius
$\underline{\underline{R}}$,	Reynolds number
u,	Tangential velocity vector
v,	radial velocity vector
V,	velocity vector
$\underline{\underline{V}}$,	Vortex number
w,	Weight per unit volume
x, y	coordinates of a point in the flow field
Y,	vertical spacing of boundaries in two dimensional flow
Γ,	Circulation
ρ,	mass density
μ,	dynamic viscosity

The differential equation which gives the pressure change normal to a streamline for flow in a curved path is

$$dp = \rho \frac{u^2}{r} dr \tag{1}$$

In the case of the irrotational free vortex, because no torque is applied to the fluid mass, Newton's second law gives $\frac{d}{dt}(ru) = 0$

$$u = \frac{K}{r} \tag{2}$$

where K is a numerical constant and shows the degree of the "vorticity". The vorticity can also be described by a characteristic, Γ, designating the velocity of rotation in the vortex thus

$$\Gamma = 2\pi r u \tag{3}$$

It can be combined with equation (2) obtaining

$$\Gamma = 2\pi K \tag{4}$$

This equation gives the relationship between the factors Γ and K, both of which are related to the degree of vorticity. Now, let us suppose that there is superposition of radial flow on this ideal vortex flow. Applying the equation of continuity to the radial flow gives

$$Q = 2\pi r Y v \tag{5}$$

where Y is the vertical distance between the flow boundaries and will be considered constant. Thus

$$v = \frac{K_1}{r} \tag{6}$$

The velocity at any point is the vector sum of the tangential and radial velocities, u and v, each of which has the same inverse relation to the radius. Thus the streamlines are equiangular and are theoretical logarithmic spirals.

Utilizing equations (1), (2), (6), and the Bernoulli theorem gives

$$\frac{p_1 - p_2}{w} = \left[\frac{K^2}{2g} + \frac{K_1^2}{2g}\right]\left[\frac{1}{r_2^2} - \frac{1}{r_1^2}\right] = \frac{K_2^2}{2g}\left[\frac{1}{r_2^2} - \frac{1}{r_1^2}\right] \tag{7}$$

showing that the radial pressure variation is hyperbolic for this idealized spiral motion between two parallel boundaries. It is to be expected that for free vortex motion through a horizontal orifice, equation (7) will be altered because of two factors. First, if the upper boundary is the free surface of the liquid, then the distance Y in equation (5) is not a constant. The depth of flow will be very nearly constant except in the region near the axis of rotation. Because of the reduction in flow area caused by the drop in the liquid surface, the velocities near the axis tend to be larger than predicted from the hyperbolic $V = \frac{K}{R}$. This is in contradiction to the effect of viscosity, however. As the velocity becomes higher in the region near the orifice, the viscous resistance becomes appreciable reducing the velocities to values less than predicted in the theoretical equation. Actual measurements show that the surface profile is above the theoretical hyperbola, as shown in Fig. 1. It must be concluded that in this region the viscous forces predominate and completely overwhelm the effects resulting from a theoretical free surface boundary.

Application of the π-Theorem

We can assume that the variables which describe free spiral vortex flow through horizontal orifices are

$$f\left[D, B, \rho, \mu, V, \Gamma, p\right] = 0 \tag{8}$$

The variables D and B characterize the boundary conditions, ρ and μ the fluid properties, V and Γ the kinematic conditions, and the surface force producing motion. According to the π-theorem[(1), (2)] there will be four dimentionless π groupings or

$$\phi\left[\pi_1, \pi_2, \pi_3, \pi_4\right] = 0 \tag{9}$$

It is assumed that D, ρ, and V are the repeating terms so that

$$\pi_1 = D^{x_1} V^{y_1} \rho^{z_1} B^{-1} \tag{10}$$

$$\pi_2 = D^{x_2} V^{y_2} \rho^{z_2} \mu^{-1} \tag{11}$$

$$\pi_3 = D^{x_3} V^{y_3} \rho^{z_3} \Gamma^{-1} \tag{12}$$

$$\pi_4 = D^{x_4} V^{y_4} \rho^{z_4} p^{-1} \tag{13}$$

It can be shown that

$$\pi_1 = \frac{D}{B} \tag{14}$$

$$\pi_2 = \frac{\rho V D}{\mu} \tag{15}$$

$$\pi_3 = \frac{V D}{\Gamma} \tag{16}$$

$$\pi_4 = \frac{\rho V^2}{p} \tag{17}$$

It will be recognized that π_1, describes the boundary proximity, π_2 is Reynolds number, π_3 is a new ratio defined here as the reciprocal of the vortex number, and π_4 is the Euler number.

Equation (9) now becomes

$$\phi\left[\frac{D}{B}, R, \underline{\underline{V}}, E\right] = 0 \tag{18}$$

Choosing the Euler number as the dependent term

$$\frac{V^2 \rho}{p} = f_1\left[\frac{D}{B}, R, \underline{\underline{V}}\right] \tag{19}$$

$$V = f_2\left[\frac{D}{B}, R, \underline{\underline{V}}\right]\sqrt{2gH} \tag{20}$$

$$Q = AV = CA\sqrt{2gH} \tag{21}$$

$$C = f_2\left[\frac{D}{B}, R, \underline{\underline{V}}\right] \tag{22}$$

Equation (22) shows that the discharge coefficient should be somehow related to the shape and character of the boundaries, the effects of viscous shear, and the effects of vorticity. It should be noted that equation (21) is the formula used for the flow through standard orifices.

Dynamic Similarity

If we consider the centrifugal force acting on a fluid element flowing in a curved path of radius, r, and tangential velocity, u,

$$F_c = \rho\, dr\, dA \frac{u^2}{r} = \rho\, dr\, dA \frac{K^2}{r^3} \tag{23}$$

The total inertia force will be

$$F_I = \rho \, dr \, dA \frac{dV}{dt} = \rho \, dr \, dA \frac{d(V)^2}{ds} \tag{24}$$

so

$$\frac{F_c}{F_I} = \frac{K^2 ds}{r^3 V dV} \tag{25}$$

which becomes dimensionally

$$\frac{F_c}{F_I} = \frac{K^2}{L^2 V^2} \tag{26}$$

showing that the vortex number

$$\underline{\underline{V}} = \frac{\Gamma}{DV} \tag{27}$$

is proportional to the square root of the ratio of the centrifugal force to the total inertia force. It is the criterion for dynamic similarity for all cases where the inertial and centrifugal forces predominate.

Laboratory Experimental Work

In 1929 studies were completed at the Technical University of Munich of the Thoma counterflow brake.(3) This device consists of a spiral vortex chamber with tangential and axial connections. Because of the tangential approach of the water and the resulting vortex, the apparatus had a great resistance to flow in one direction, while the flow in the opposite direction encountered far less resistance. The apparatus was considered as a type of pipe fitting which might replace the check valve in special cases, and the changes in the fitting resistance factor for various designs in the two-directions of flow, were studied.

Studies of free vortex flow through orifices were completed just prior to 1950 at the State University of Iowa.(4) Tests were made on a 4-inch sharp edge circular orifice in the center of the bottom of a 6-foot diameter tank. Tangential flow was directed into the tank trough four 1-inch pipes equally spaced around the circumference and radial flow was guided in through a false bottom directly below these jets. A constant head of 1.63 ft. was maintained with the variables being the tangential and radial approach velocities. Attempts were made to show the normal orifice coefficient of discharge as a function of the ratio of the average tangential component of velocity to the average radial component of velocity.

During this same period studies were made at the University of Wisconsin(5) of the flow in centrifugal pressure nozzles to be used as atomizers. The atomization is caused largely by a vortex motion imparted to the liquid as it passes through the nozzle. A common method of imparting this vortex motion to the liquid is to provide a chamber within the nozzle with a tangential inlet. The characteristics which were studied in this investigation were the nozzle spray cone angle and the discharge coefficient for various nozzle designs.

The need for further fundamental research on this subject was recognized by the senior author who consequently urged an experimental program to be commenced at the Hydraulics Laboratory of the University of Wisconsin, the objectives of which were to extend the fundamental and practical knowledge of the spiral vortex and in particular to find the head-discharge relations for a series of orifices with a variety of boundary conditions.(6)

This experimental work consisted of tests made in two different tanks. The initial tests were made in a 12-foot diameter tank with sheet metal sides, 2 feet high. Orifices were cut in 2-foot diameter, 14-inch gage sheet metal plates and were placed in the center of the bottom of the tank. Water was admitted to the tank at four points around its periphery and allowed to approach the orifice uniformly after passing through a rock baffle ring. The discharge was measured by the direct volumetric method. Static head was measured at the third points of the periphery by three point gages located about three inches in from the rock baffle ring. Vortex profile measurements were taken with a moving point gage located on a 6-inch structural aluminum I-beam 12 feet long. An angle mount was fastened to the center of the beam to support a special inside caliper designed by the senior author, and donated by Leupold & Stevens Instruments Inc. of Portland, Oregon, which could be lowered into the vortex air core to measure its diameter.

The vortexes which formed during this stage of the testing, with uniform approach conditions, were small and unstable. The direction of rotation would change from clockwise to counter-clockwise with no definite pattern and the air core would form only momentarily. These vortexes were found to have no noticeable effect on the standard orifice coefficients.

In order to induce greater degrees of vorticity a 4.5 foot diameter ring with 19 guide vanes was constructed concentric with the orifice opening. The vanes were 1 foot wide and 1.5 feet high and were evenly spaced around the circumference of the ring. They were so arranged that each could be pivoted about its center or bolted shut to the adjacent vanes. With the aid of these vanes the water could be directed toward the orifice opening with any desired approach angle.

When the water was allowed to pass through the vanes with a velocity component tangential to the orifice, a stable vortex formed and centered itself over the orifice opening. Tests were made with all combinations of vane openings from all vanes open tangential to the orifice opening to one vane open and all the others closed. The type of entrance did not appear to have any effect on the stability or the geometry of the vortex so long as the entrance velocity had a tangential component.

A second series of tests were made in a 6 foot diameter steel tank, 3 feet high, in order to extend the data with respect to head and boundary proximity. The orifices were cut in 18-inch diameter, 14-gage sheet metal plates and were fastened to the tank in the geometric center. Two inner boundary rings were used in this tank. Each was constructed of 16-gage sheet metal with two diametrically opposite vanes which could be pivoted to any desired entrance angle. The larger boundary ring was 4.5 feet in diameter and 2.5 feet high while the smaller ring was 2 feet in diameter and 3 feet high.

No baffles were provided for the initial tests on this apparatus. The water was allowed to approach the inner boundary ring with an initial vortex motion imparted to the water admitted through six 2-inch pipes, 6-inches on centers welded into the tank tangentially at each of two diametrically opposite points. At the higher flow rates, large eddies were formed behind the entrance vanes which greatly disturbed the surface both inside and outside the boundary ring. Because of this, deflector vanes 6 inches wide and 3 feet high were fastened in front of the discharging jets, which produced a smooth water surface condition.

The circulation was determined from the water surface profile measurements, by using equation (4) and the theoretical equation for the water surface hyperbola.

$$y = \frac{K^2}{2gx^2} \tag{28}$$

Because the surface curve does not conform exactly to this hyperbola, the measurement of the drop down, y, was standardized at a point x = D for each test run. Above this point, the theoretical and actual curves were in close agreement.

Figs. 2 to 6 show the relationship between the orifice coefficient, C, and the vortex number $\underline{\underline{V}}$ (equation 27). This equation may be written

$$\underline{\underline{V}} = \frac{\Gamma}{D\sqrt{2gH}} = \frac{2\pi x\sqrt{2gy}}{D\sqrt{2gH}} \tag{29}$$

notice that when $X = \frac{D}{2}$

$$\underline{\underline{V}} = \frac{\pi\sqrt{2gy}}{\sqrt{2gH}} \tag{30}$$

This shows that when the vortex number equals π, the drop down, y, at the edge of the orifice must equal the static head, hence the coefficient C must be zero. This point seems well substantiated by the test data.

Between V = 0.8 and V = 3.14 = π, the straight line equation

$$C = 0.686 - 0.218\,\underline{\underline{V}} \tag{31}$$

is proposed to determine the approximate coefficient for vortex flow through horizontal sharp-edged orifices. Between V = 0 and 0.8 the coefficient can be estimated from the curve, or computed from (31). The smallest ratio B/D tested was 4.0. With this boundary proximity, the water surface profile still conformed to the theoretical hyperbola above the point x = D. With the small boundary ring, however, the asymptote of the water surface curve was not reached so that in order to calculate the circulation it was necessary to solve (28) for two points on the curve simultaneously, which amounts to an extrapolation for the static head, H.

The Portland Interceptor Diversions

There has been developed a number of devices for controlling the flow diverted into interceptors from combined sewers.[8] Practically all such devices have moving parts and as such are subject to heavy maintenance and occasional stoppage from solids carried by sewage. Perhaps the most successful of such devices, which, however, is only practicable in large sewers, is the motor operated gate, controlled by a float.

In the Portland sewerage system only one such gate device was installed. This was on a 96-inch sewer with a 54-inch interceptor. All of the other diversions were effected by vortex control. A typical diversion structure is shown in Fig. 7.

A small dam was built in the sewer at the downstream side of the diversion manhole. It has faces of 1 on 2 slopes which permit the passage of grit down stream. The height of these dams is generally about 10% greater than the normal depth of the sanitary flow in the sewer. They must be high enough to divert all the sanitary flow through the orifice. Beyond this, local conditions such as slope of the main sewer, etc., governed the height of the dams. Fig. 8 gives the details of the diversion dams.

The vortex chamber is U-shaped with the orifice in the center of the arc and the open portion tangent to the sewer. When the upper portion of the sewer was removed for the construction of the diversion manhole, a low lip or weir was left to aid in keeping grit from getting into the interceptor.

Orifices were cut in steel plate that fitted into a shallow recess made by brazing a ring around the flange of the cast iron entrance elbow of the interceptor (See Fig. 7).

Design Criteria

In order to obtain flow data for each combined sewer of the city, and beginning well before the design contract for the Sewerage project was executed, 42 out of a total of 54 sewers that flowed into Willamette River or Columbia Slough were selected for continuous measurements of sewage and storm water flow during 1941-43. Knowing the residential population and the character and magnitude of industrial development tributary to each selected sewer or portion thereof, certain criteria were ascertained from the sewage measurements that could be expanded to approximate the sanitary and normal flows of each of the 160 diversion structures from various points on the interceptors that had to be made.

Table I shows how the design criteria thus obtained was applied to a few sample diversion structures.

Table I Design Criteria for Typical Diversions

Main Sewer		Diversion		Flow cfs		Orifice		Sanitary	Vortex	Dam
Size inch	Slope	Symbol	Size inch	Design	Sanitary	Elev ft	Diam inch	Depth ft	chamber Diam ft.	hgt inch
18	.048	SJ17	8	0.46	0.13	74.7	6	0.26	3.0	5
8	.072	SW72	8	0.13	0.06	55.5	6	0.16	3.0	4
22	.013	SE119	12	0.35	0.09	33.0	8	0.29	4.0	5
36	.002	SW21	15	1.15	0.62	28.8	12	0.59	4.5	13
36x36	.069	NW9	15	3.57	2.84	41.9	12	0.73	4.5	13
48	.031	CE12	18	2.62	1.26	52.5	16	0.80	5.0	12
42	.024	SW6	21	4.95	3.08	22.5	18	0.95	5.0	13
62	.02	EC93	24	4.80	2.07	54.0	20	0.95	5.0	13
87	.0034	EC127	27	7.44	3.14	159.8	24	1.09	5.0	13
48x72	.022	WC100	24	9.61	6.20	25.5	20	1.20	5.0	16
72x80	.009	WC7	24	10.7	5.83	25.0	20	1.24	5.0	16
54x54	.022	NE55	30	10.5	2.72	51.2	28	1.24	5.0	17
66x72	.014	EC109	36	7.52	4.03	56.0	33	1.07	6.0	14

Model Studies of Diversion

In addition to the laboratory experiments heretofore described, a model of a diversion vortex chamber in which the orifice sizes could be changed was built and tested in a local laboratory belonging to the Northwest Machine Works, used primarily to determine the behavior of small farm turbines. This plant had all the necessary facilities for the tests desired to be made.

These experiments produced valuable data on the vortex flow through horizontal orifices 3, 4, 5 and 6 inches in diameter under both free flow and submerged conditions; desirable sizes of the vortex chamber; the proportion of grit that could be kept out of the interceptor and other needed information.

Data on grit was determined by introducing known weights of coarse sand

and small gravel into the sewer and then weighing the quantity that passed through the orifice and down the sewer. For the 3-inch orifice 90% of the grit introduced passed over the dam and on down the sewer. For the 5-inch orifice 87% of that introduced continued on down the main sewer. The grit passed through the orifices was of the smaller sizes, mostly having been thrown into suspension upstream of the diversion. The coarser formed a blanket just upstream of the dam and was later carried over the dam during higher flows.

Coefficients of Flow

For 1-foot heads under vortex flow the coefficient C in (21) was 0.49, 0.39 and 0.36 for 3, 4 and 5-inch orifices respectively. These are not as low as those obtained under higher heads at the University of Wisconsin laboratory (see Figs 3 and 4) and of course, not as low as actually obtained under the higher heads during heavy storms.

Fig. 9 shows the model of the vortex chamber under test. The vortex is free, i.e., there is no external force applied and it is completely aerated through the orifice.

The interesting flow relationships of such a vortex are well illustrated in Fig. 10, for the model with a 4-inch orifice. Note that at a 1-foot head the flow in the sewer was 2.7 cfs. while the flow passing through the orifice was only 0.3 cfs. or 11% of the sewer flow.

Automatic Shut-Offs

Two devices were tested in the laboratory to shut off the interceptor entirely during heavy storms. The effect of this is to compensate for the increased flow to the treatment plant from sewers having only vortex control at such times in the smaller diversions, and thus not overload the treatment plant. The device that appeared the most practical is known as the Cone Gate, shown in Fig. 11.

The stem of the gate is supported on a pair of hinged bars or links so that the gate always moves parallel to itself. The cone is normally held well above the orifice. This passes practically all debris and does not interfere with the vortex. The cone is enclosed top and sides by brass plate. Between the cone and the sides is a horizontal screen forming a ring through which water can enter and fill the brass cone.

The cone stem is normally held against a stop by a small cable passing over overhead pulleys and counterweighted. When storm water appears and rises enough to pour through the screen and fill the cone the increased weight overcomes the counterweight and the cone seats itself in the orifice shutting off the flow into the interceptor.

A hole in the bottom of the cone permits the cone to empty as the storm subsides, whereupon the cone is automatically raised against the stop by the counterweight, while any solids caught against the screen fall off.

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7. The design and advisory supervision of the Portland Sewerage Project was done jointly by Stevens & Koon (predecessors to Stevens & Thompson) and J. W. Cunningham & Associates. A. W. Rawn, Mem. ASCE, was Consulting Engineer.

8. Handbook of Applied Hydraulics, Second Edition p. 1066, Calvin Davis, Editor-in-Chief, McGraw-Hill Book Company, Inc., New York.

ACKNOWLEDGEMENTS

The experimental work done at the University of Wisconsin Hydraulics Laboratory by the junior author was under the direction of Dr. James R. Villemonte. The Department of Civil Engineering through the interests of Dr. J. G. Woodburn, Chairman, and Dr. A. T. Lenz, Laboratory Director gave complete support to the work.

Financial support for the project was given by the Research Committee of the Graduate School from funds supplied by the Wisconsin Alumni Research Foundation and the senior author.

Messrs. B. E. Hirsch, T. J. Kontos, and D. W. Hinderman, Civil Engineering students, gave considerable assistance in the construction of the apparatus and the collection of data.

Leupold & Stevens Instruments, Inc., of Portland, Oregon donated to the University of Wisconsin Hydraulic Laboratory a gage that proved to be a very valuable piece of equipment for measuring the interior diameter of the Vortexes.

S. Ben Morrow, former Water Superintendent and City Engineer of Portland, rendered whole hearted cooperation and assistance for the Vortex model studies and for all other phases during the design and construction stages of the Portland Sewerage Project.

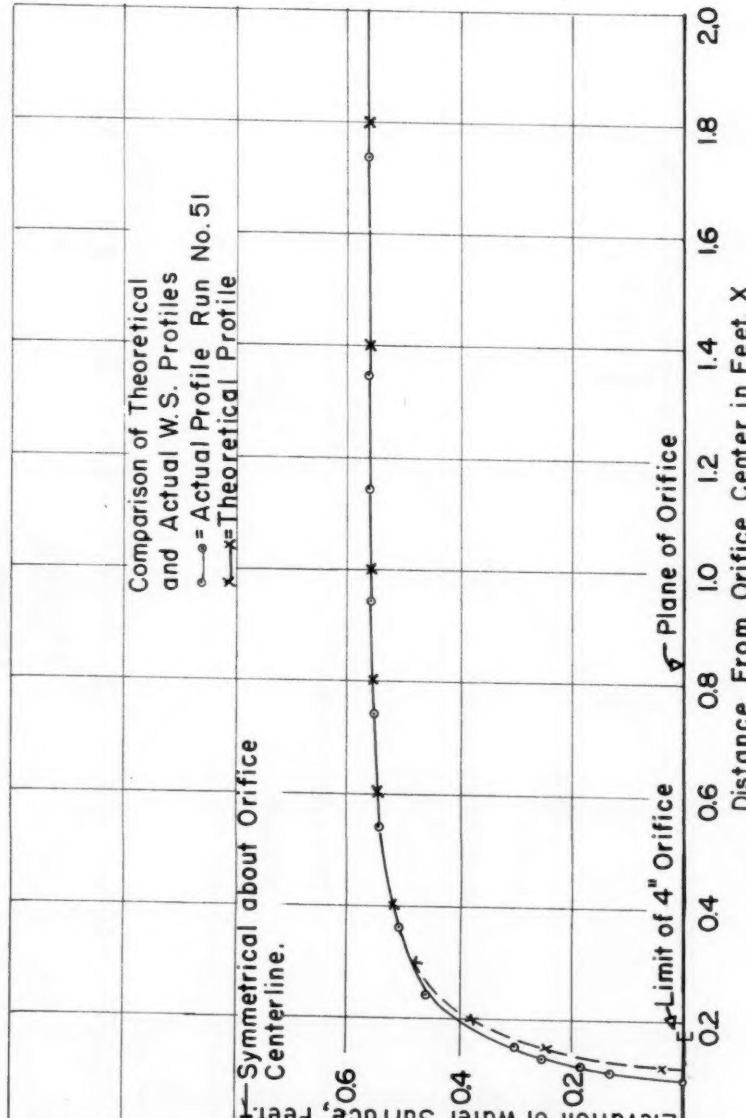

Comparison of Theoretical and Actual W.S. Profiles
= Actual Profile Run No. 51
= Theoretical Profile
Symmetrical about Orifice Centerline.
Limit of 4" Orifice
Plane of Orifice
Elevation of Water Surface, Feet.
0.6
0.4
0.2
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Distance From Orifice Center in Feet, X

FIGURE 1.

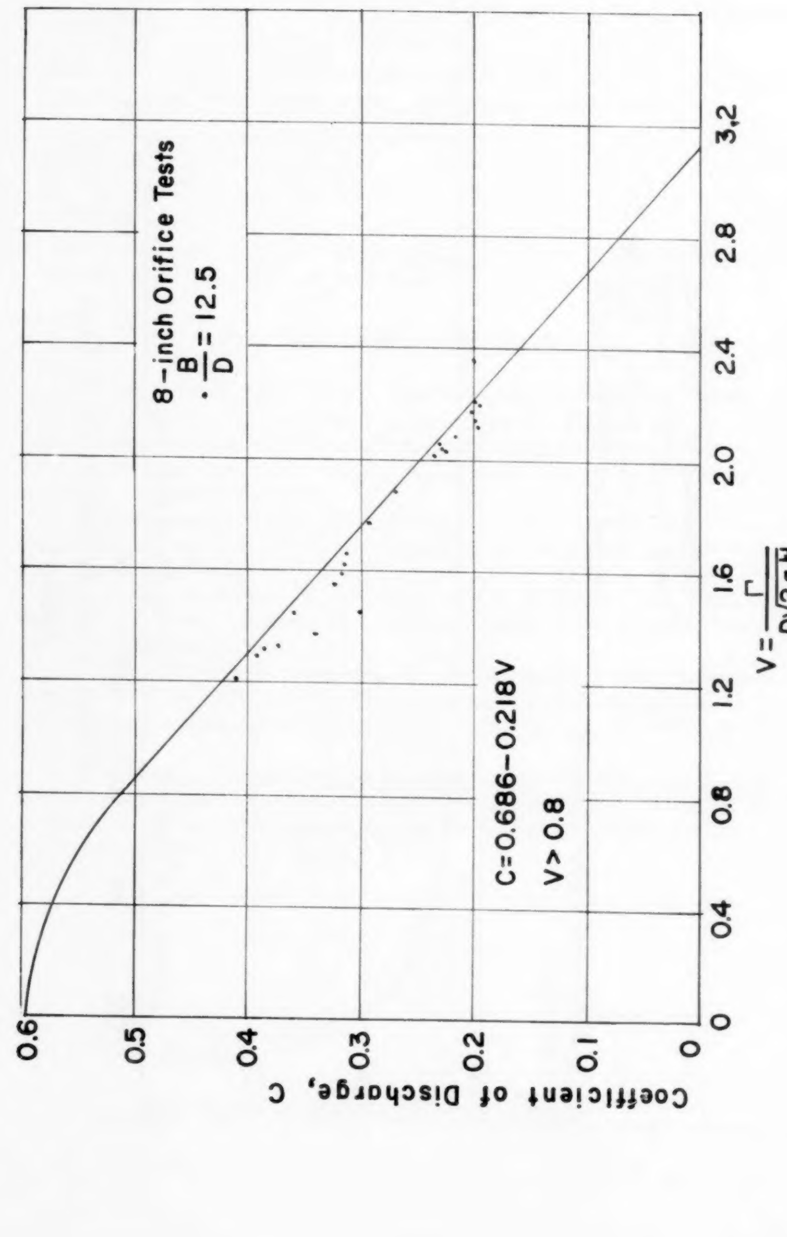
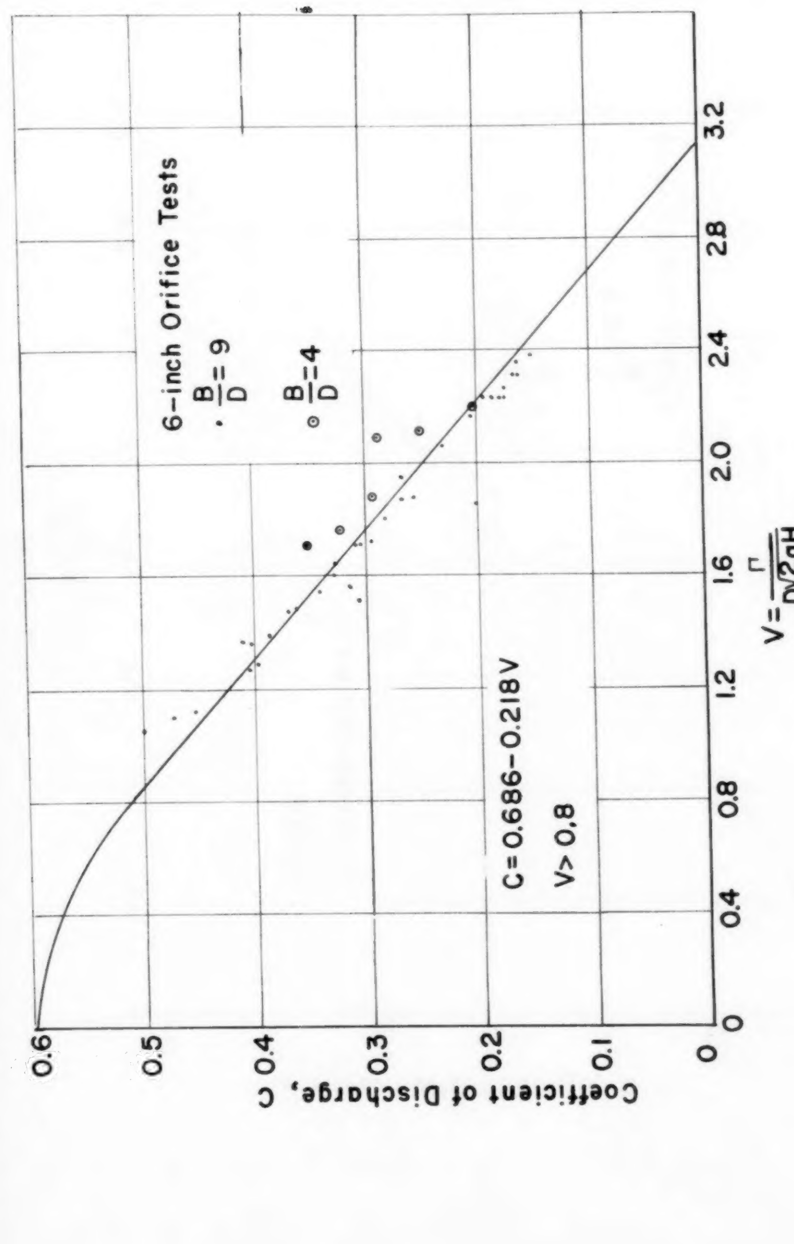

8-inch Orifice Tests
$\frac{B}{D} = 12.5$
$C=0.686-0.218V$
$V > 0.8$
$V = \frac{r}{D\sqrt{2gH}}$
Coefficient of Discharge, C
0
0.1
0.2
0.3
0.4
0.5
0.6
0.4
0.8
1.2
1.6
2.0
2.4
2.8
3.2

FIGURE 2



6-inch Orifice Tests
$\frac{B}{D} = 9$
$\frac{B}{D} = 4$
$C = 0.686 - 0.218V$
$V > 0.8$
$V = \frac{r}{D\sqrt{2gH}}$
Coefficient of Discharge, C
0 0.1 0.2 0.3 0.4 0.5 0.6
0 0.4 0.8 1.2 1.6 2.0 2.4 2.8 3.2

FIGURE 3

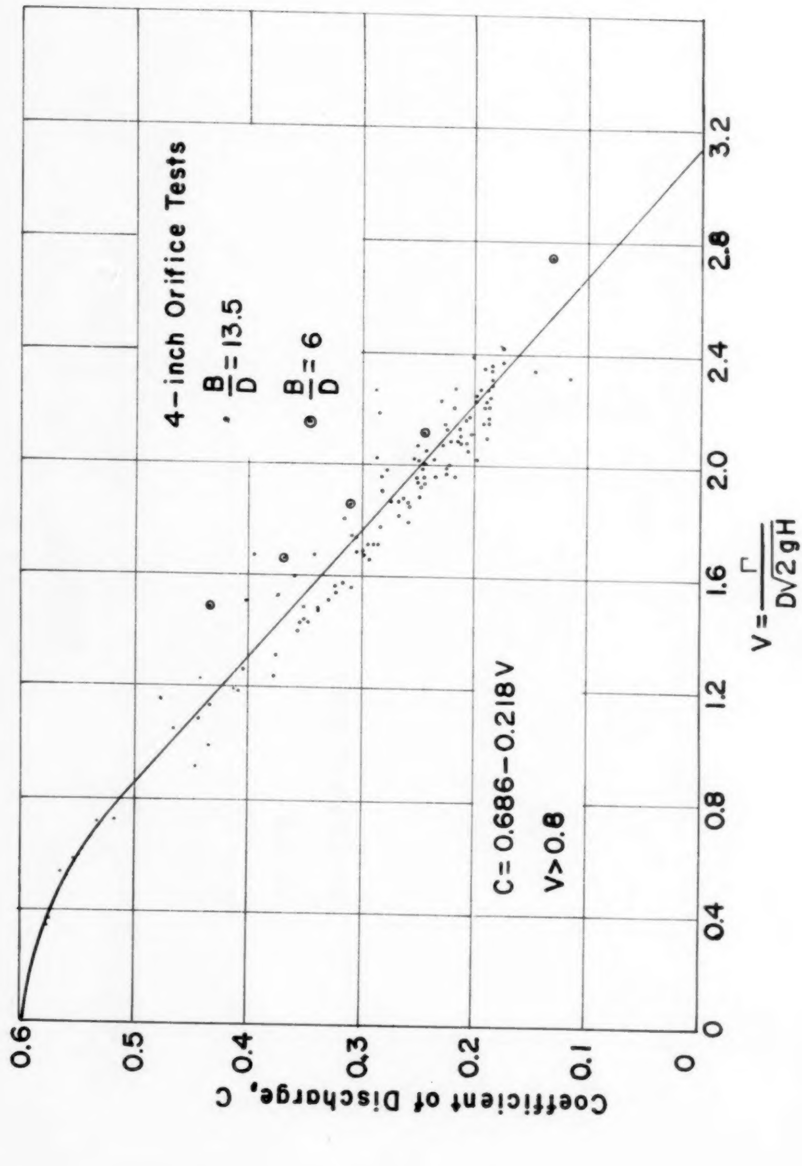

4-inch Orifice Tests
B/D = 13.5
B/D = 6
C = 0.686 − 0.218V
V > 0.8
Coefficient of Discharge, C
0.6
0.5
0.4
0.3
0.2
0.1
0
0.4
0.8
1.2
1.6
2.0
2.4
2.8
3.2
V = Γ / (D√2gH)

FIGURE 4

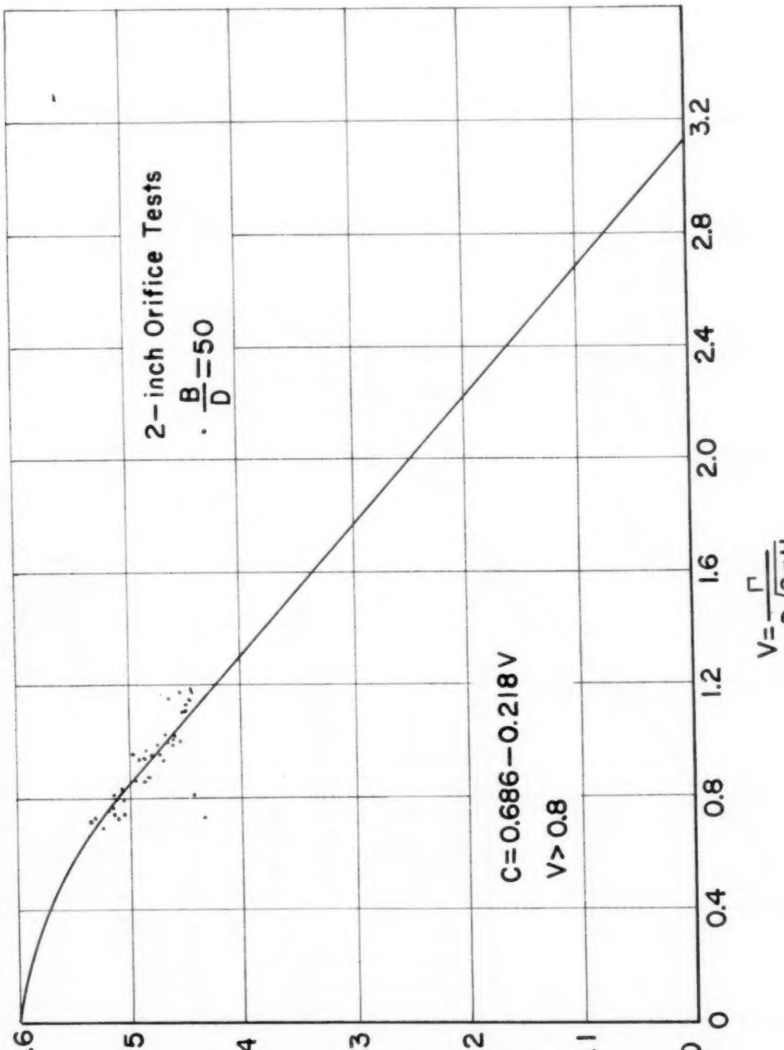

2-inch Orifice Tests
$\frac{B}{D} = 50$
$C = 0.686 - 0.218V$
$V > 0.8$
$V = \frac{\Gamma}{D\sqrt{2gH}}$
Coefficient of Discharge, C
0
0.1
0.2
0.3
0.4
0.5
0.6
0
0.4
0.8
1.2
1.6
2.0
2.4
2.8
3.2

FIGURE 5

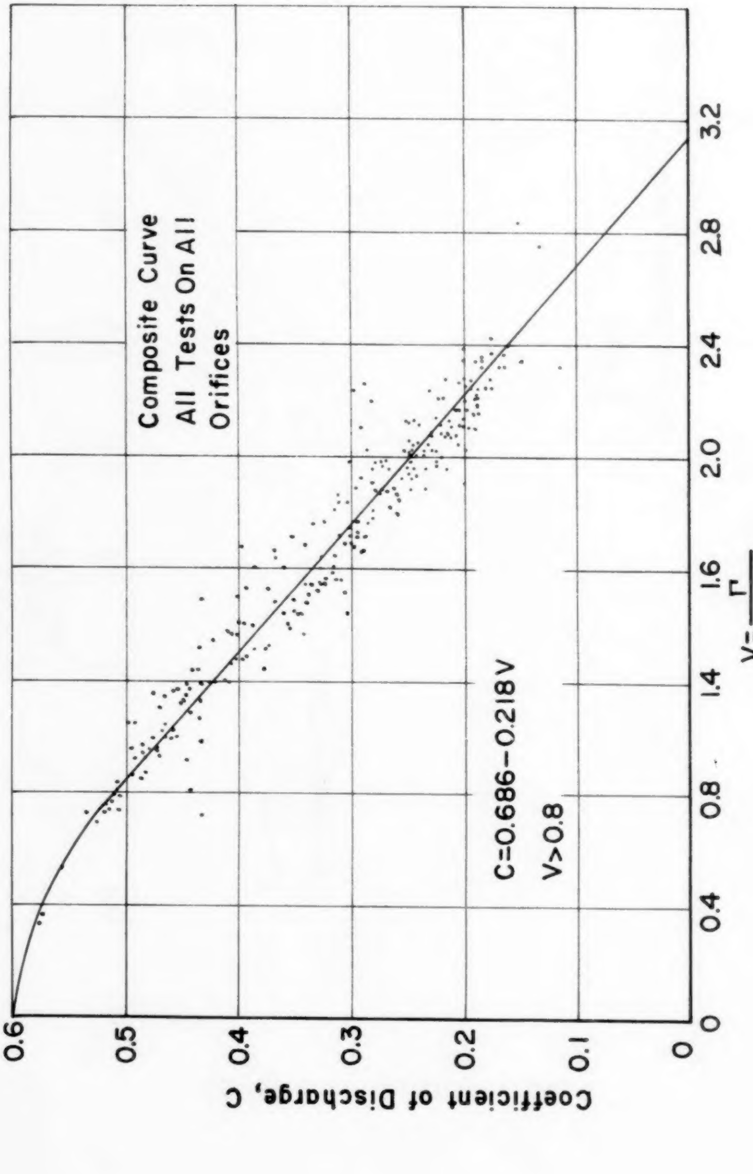
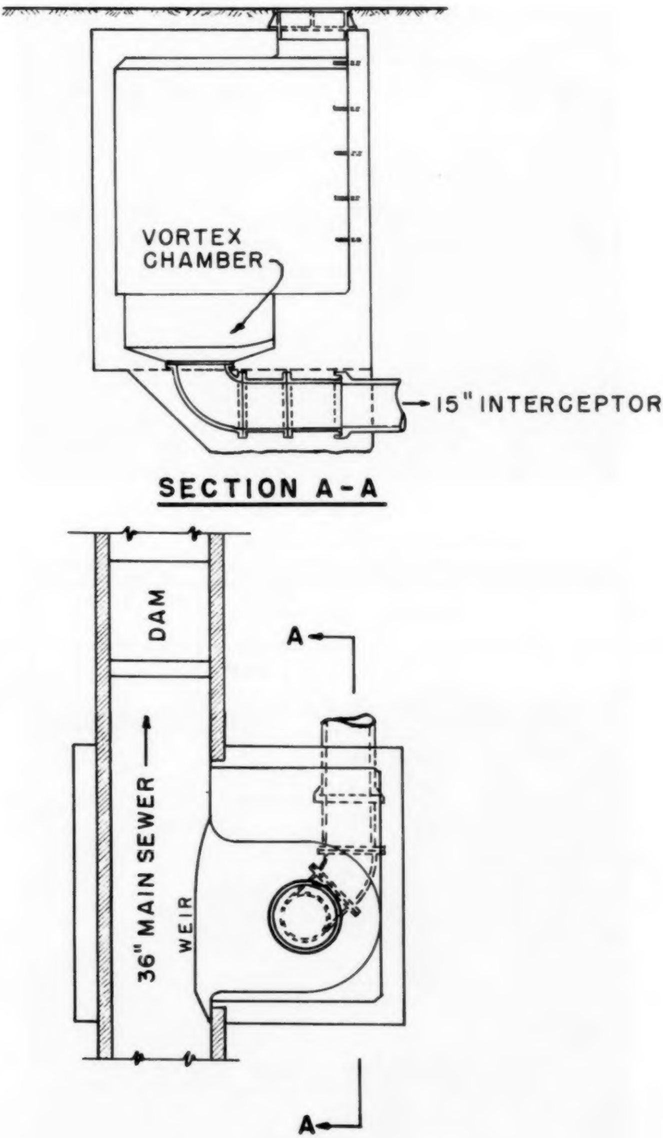

Composite Curve
All Tests On All
Orifices
C=0.686-0.218V
V>0.8
Coefficient of Discharge, C
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.4
0.8
1.2
1.6
2.0
2.4
2.8
3.2
V=Γ/(D√2gH)

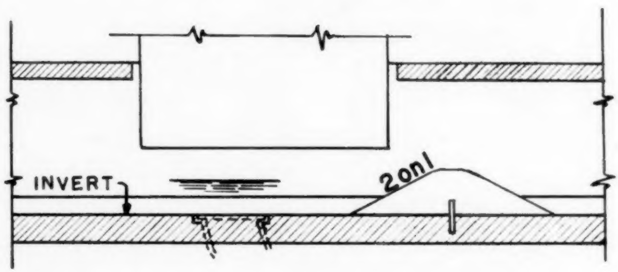
FIGURE 6



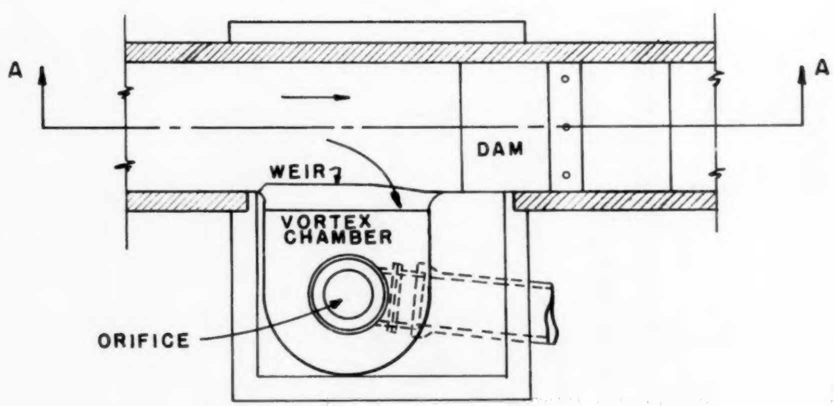
TYPICAL VORTEX DIVERSION

NO SCALE

Fig. 7



SECTION A-A



PLAN

DIVERSION DAM DETAILS

NO SCALE

Fig. 8

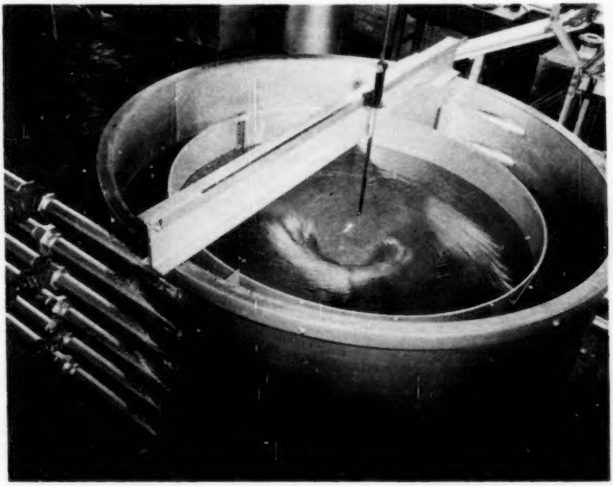


Fig. 9a
12-Foot Vortex Tank -- University of Wisconsin Hydraulic Laboratory

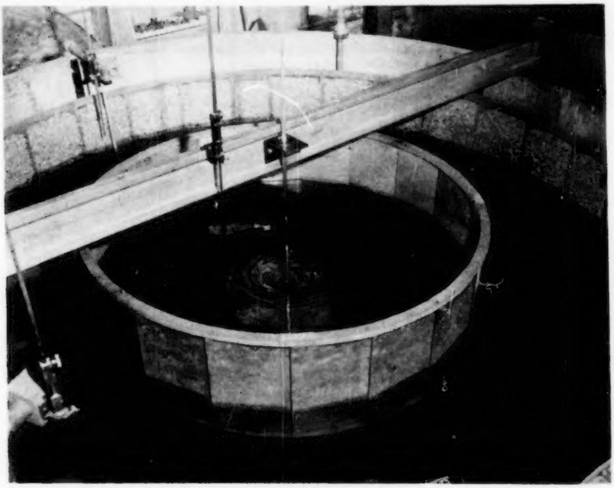


Fig. 9b
6-Foot Vortex Tank -- University of Wisconsin Hydraulic Laboratory

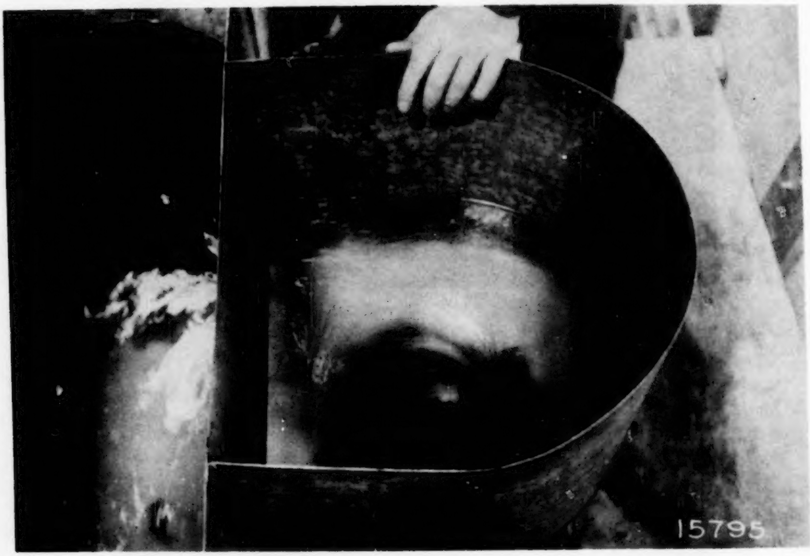


Fig. 9c
Model of Vortex Chamber Under Test
Portland Sewerage Project

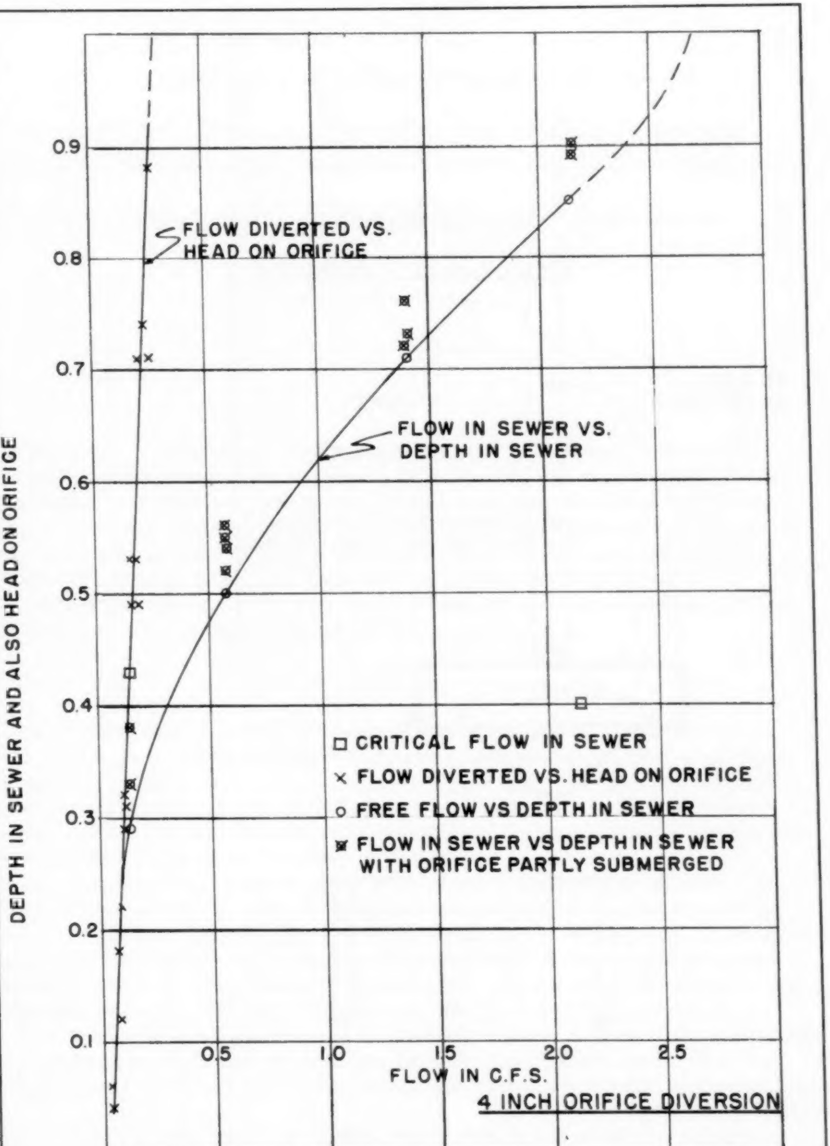

DEPTH IN SEWER AND ALSO HEAD ON ORIFICE
FLOW IN C.F.S.
FLOW DIVERTED VS. HEAD ON ORIFICE
FLOW IN SEWER VS. DEPTH IN SEWER
CRITICAL FLOW IN SEWER
FLOW DIVERTED VS. HEAD ON ORIFICE
FREE FLOW VS DEPTH IN SEWER
FLOW IN SEWER VS DEPTH IN SEWER WITH ORIFICE PARTLY SUBMERGED
4 INCH ORIFICE DIVERSION

Fig. 10

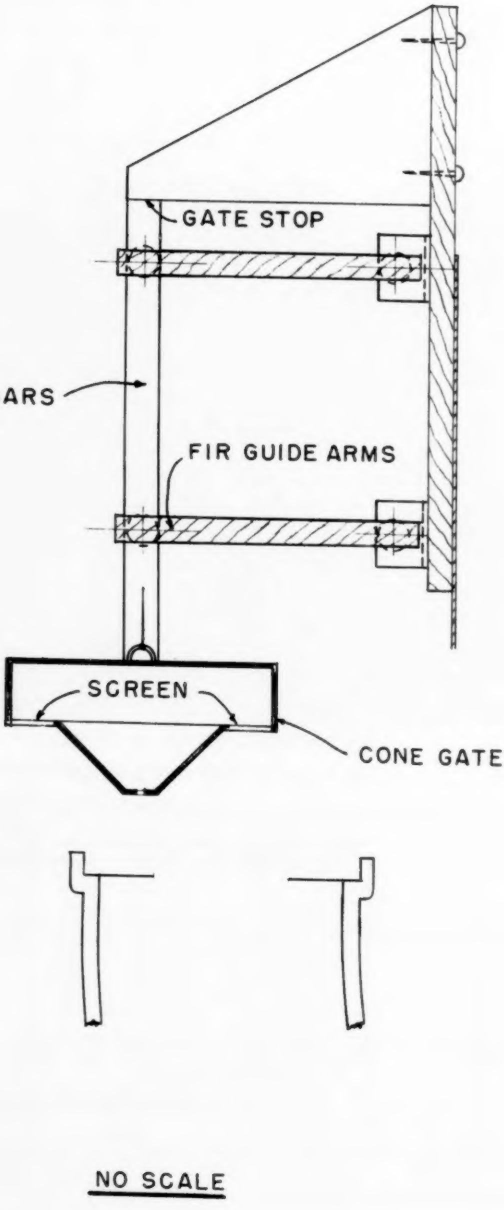

GATE STOP
BRASS GUIDE BARS
FIR GUIDE ARMS
SCREEN
CONE GATE
NO SCALE

Fig. 11

Paper 1462 December, 1957 SA 6

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

FISCAL OPERATIONS OF THE BUFFALO SEWER AUTHORITY[a]

Frederick W. Crane[1]
(Proc. Paper 1462)

SUMMARY

This paper is an attempt to describe the fiscal phases of operation of a metropolitan sewerage system under a public authority organization. Public authorities have become quite prevalent and much has been said both in their favor and in their disfavor. One criticism is their remoteness from political response. One of their virtues is this same remoteness from political involvement. It is probable that they are not an ideal interpretation of our democratic principles of government, but they are a means for progressive and business-like accomplishment.

The Buffalo Sewer Authority is one of the oldest public authorities in the field of sewage disposal. It was created by enactment of the New York State legislature in April 1935 for the purpose of providing "an effectual means for relieving the Niagara river, Buffalo river and Lake Erie from pollution by sewage and waste" and "to maintain and operate a comprehensive system for the disposal of sewage and waste."

The Buffalo Sewer Authority is a public benefit corporation directed by a board of five members, each of whom is appointed by the Mayor of the City of Buffalo for 3-year terms. The board members serve without pay. The board employs a General Manager as administrative head of the Authority and an operating force of approximately 240 persons. All operating personnel of the Authority other than the General Manager are civil service employees.

The Authority act was designed to integrate the operations of the Authority with those of the City and under it the Comptroller, the Treasurer and the Corporation Counsel of the City serve in like capacities for the Authority.

Note: Discussion open until May 1, 1958. Paper 1462 is part of the copyrighted Journal of the Sanitary Engineering Division of the American Society of Civil Engineers, Vol. 83, No. SA 6, December, 1957.

a. Presented at Convention of American Society of Civil Engineers, Buffalo, New York. June 6, 1957.

1. General Manager, Buffalo Sewer Authority, Buffalo, N. Y.

The City has always fully cooperated with the Authority and has permitted, primarily for the benefit of the taxpayer, use by the Authority of facilities and services of the Department of Assessment, the Department of the Treasury, the Department of Law, the Department of Audit and Control, the Division of Water, and the Division of Purchase of the City.

All moneys paid to the Authority are received by the City Treasurer as agent for the Authority and are deposited by him in banks or trust companies who continuously secure such accounts by pledge of direct obligations of the United States of America or of the State of New York having an aggregate market value, exclusive of accrued interest, equal at all times to the amount of deposit. Disbursement of Authority moneys is made upon warrant of the City Comptroller, as agent for the Authority, after audit by him and on requisition of designated persons in the Authority.

The first task of the Authority when it came into existence in 1935 was to construct a sewage treatment plant and the necessary system of intercepting sewers. This was done under the direction of a firm of consulting engineers and was financed under P.W.A. auspices by the sale of bonds and by federal grant. Upon certifying to the Common Council of the City and to the Secretary of State of the State of New York that this work had been completed, the then existing sewer system of the City, together with all records pertaining to it, was transferred to the jurisdiction of the Authority, and it now has complete control and possession of the system and all facilities in the city for the disposal of sewage and storm water.

The Authority was authorized to establish a schedule of rents or charges to be called "sewer rents" uniformly applicable to all real property served by its facilities. Such rents were to be based upon any equitable basis as determined by the Authority, such as the consumption of water, the number and kind of plumbing fixtures connected with the Authority's facilities, or the number of persons served. Provision was made for publication and for inspection for a period of 30 days of any proposed schedule of sewer rents or of any modification. In the 19 years that sewer rents have been in effect, there have been no objections filed although the schedules have been modified at least ten times.

With the broad bases for sewer rental as allowed by the legislative act, various methods of levying the same were examined and that which seemed the most equitable and most practicable of application was adopted. This determination was complicated by the fact that the Authority's revenues have to support its full operations including the construction, maintenance and operation of a sewer system as well as a sewage treatment plant, and the sewer system being a combined one is a charge against both sanitary waste and storm water.

There appears to be general acceptance of the theory that the proper distribution of the cost of handling and treating sewage (exclusive of industrial waste) is on the basis of water use. However, the Authority has other costs that are of general benefit and not properly proportioned to water use. These latter costs are those required for the handling of storm flows and to a lesser extent those necessary to provide capacity in plant and sewer system for future development and growth. The Authority's basic conclusion was and has remained that it is unfair to place the entire cost of a broad sewage and storm water disposal operation on water users and equally unfair to impose the full cost on real estate owners.

The adopted rental schedule of the Authority, therefore, provides generally

for two principal sources of revenue; one being from rental based on water use and the other from rental based on assessed valuation of taxable real estate. The required total annual revenue is divided on a calculable basis and in early operations 55 per cent was obtained from rental based on water use and 45 per cent from rental based on real estate valuation. The relatively heavy storm water relief program upon which the Authority has been engaged throughout its existence has created increasingly heavy annual debt expense that has caused a reversal in proportionate revenues so that at present approximately 40 per cent of revenue is obtained from rental based on water use and 55 per cent from rental based on real estate valuation. Experience has thus altered the price policy of the Authority.

Other miscellaneous revenues make up the remaining 5 per cent of total revenues. The above revenue ratios do not always accurately reflect the true division of benefits because of such practicable considerations as the simplicity in making changes in rental rates on assessed valuations in comparison with the major complexity and cost of changing rental rates on water use.

Early in the Authority's operations great concern was expressed by railroads and other industrial users of large quantities of water over sewer rental charges on water used and not discharged to the facilities of the Authority. Every complaint of such circumstance was investigated and total or partial exemption made as the case may indicate. It was found that a high degree of exactitude in such determinations was generally economically undesirable. Allowances for water used in lawn sprinkling or similar seasonal evaporative operations are determined by comparing seasonal averages.

Water obtained from sources other than the City's water supply is charged for in the same manner as for City water. The most common of such sources is well water supply that industries and others find advantageous for cooling purposes. The Authority requires that this water be measured and its high sulfur content makes meter maintenance a major problem. It is of interest that while it is not too difficult to obtain an ample supply of water from wells in the Buffalo area, it is next to impossible to return any appreciable amount of this water to the ground. One case, however, exists in the City of an industrial establishment that discharges a large proportion of its consumed water into an abandoned and dry gas well of considerably greater depth than a normal water well. An allowance is made in sewer rental charge for such disposal.

A number of industrial wastes discharged to the sewer system are of such nature as to increase costs of treatment. This was recognized in the early days of the Authority and special charges were established for the treatment of such wastes. Such charges are now regularly made in a number of instances and are intended to reimburse the Authority for the cost of chlorine, chemicals, and power used in excess of normal sewage requirements. Oxygen demand is not much of a factor when you have the Niagara River with 220,000 cfs flowing past your doorstep.

The Authority adopted the following formula as a means of simple application of such special charges:

$$R = FP_c\,(C - Nc) + FP_s\,(S - Ns)$$

R is the rate of special charge per 1,000 cu. ft. of volume of waste

$FP_c\,(C - Nc)$ represents the cost of the excess chlorine demand of a waste under consideration and is derived as follows:

Nc is the normal chlorine demand of sewage.

C is the measured chlorine demand of the waste under consideration.

Therefore, (C - Nc) is the excess chlorine demand of the waste.

F is a conversion factor to convert chlorine demand from the usual laboratory measurement of parts per million to the more common usage of lbs. per 1,000 cu. ft.

Pc is the cost of chlorine.

FP_s (S - Ns) represents the cost of chemicals and power required for conditioning and disposal of excess solids in the waste under consideration and is derived as follows:

Ns is the normal quantity of suspended solids in sewage.

S is the measured quantity of suspended solids in the waste under consideration.

Therefore, (S - Ns) is the excess quantity of suspended solids in the waste.

F is the same conversion factor as described above.

Ps is the cost of chemicals and power as determined from plant operations.

In all cases, payment of industrial waste charges has been made promptly. The cost of sewer cleaning attributable to industrial waste blockages is directly charged against the responsible concerns.

When delinquent, sewer rental charges constitute a real property lien having the same priority and superiority as the lien of the general tax of the City of Buffalo. The sewer charge, if delinquent more than 90 days, may be immediately foreclosed by the Authority in the Supreme Court of Erie County, New York. This is a much more effective method than the procedure for collection of general city taxes, which requires a sale of the tax certificate, and a two-year period for redemption before foreclosure.

All operating expenditures of the Authority are controlled by budget adopted annually prior to the commencement of the fiscal year. The budget has been of the line-by-line type with balanced expenditures and revenues. The balancing item has been revenue expected from sewer rent based on assessed valuation of real property because as previously mentioned this is the more flexible part of the income producing structure. The aggregate amount required for this item is spread over the city's assessment rolls and results in a rate per $1,000 assessment value. The present rental rate is $1.517+ per $1,000 assessed value.

Sewer rent based on assessed valuation is billed on a separate and distinct bill, covering a fiscal year, which is folded in with and accompanies the city tax bill for each parcel of property. These rental bills are prepared under agreement by the City Department of Assessment and by the International Business Machines Corporation.

Sewer rent based on water use is billed as a separate and distinct item on each water bill by mechanical systems installed at the joint expense of the City Division of Water and the Authority. Much of Buffalo's water billing is on a flat rate basis and sewer rental billing in such cases is greatly repetitive from period to period. Where billings are based on quantity of consumption,

such quantities are determined by the Division of Water and rentals are computed in that Division.

Collections of sewer rent are handled by the City Department of the Treasury and are uniformly excellent averaging better than 96 per cent. Foreclosure actions have not so far been undertaken in view of the good collection record and the small amounts involved in the individual delinquent accounts. Properties delinquent in payment of sewer rents are usually also delinquent in payment of general city taxes. The City in foreclosing the lien of its taces has the amount and priority of the lien of sewer rent determined and the latter is paid out of the proceeds of the sale. The Authority, the City of Buffalo and the County of Erie participate proportionately in the proceeds from rental or sale of property acquired at tax foreclosure sales or by voluntary conveyance in lieu of tax foreclosure.

The Authority serves small areas outside the city limits. The basic charge for such service as set forth in the Authority's Schedule of Sewer Rents is at rates approximately double those charged within the City. Where such areas constitute districts in the adjoining townships, agreements have been entered into providing for a charge to the town based on actual costs as determined by apportioning the total capacity of the connections with the Authority's system to the total capacity of the sewage treatment plant of the Authority. These agreements are mostly for a period of 20 years and are subject to a permissive review of costs at 5 year intervals.

As previously mentioned operating expenditures are controlled by a budget adopted annually. The Authority also controls its construction or capital expenditures by budget which is adopted for six month periods commencing January 1st and July 1st. Because of variations that inevitably occur between budget estimates and actual costs and in order to maintain some flexibility in scheduling smaller projects particularly, a budgetary period of six months has been found advantageous.

Construction funds are obtained from bond sale. All bonds of the Authority are revenue bonds based upon sewer rentals and other revenues. The Authority has power to issue its negotiable bonds for any of its corporate purposes provided the total amount "issued and outstanding at any one time shall not exceed in the aggregate the sum of $15,000,000." The Authority has issued and sold a total of $18,865,000 in bonds and has as of June 1st of this year retired $6,570,000 leaving a total of $12,295,000 presently outstanding. A large local trust company acts as fiscal agent for the Authority in all matters relating to the Authority's bonds.

The most recent bonds sold by the Authority are its Series "O" or 15th issue, sold June 1, 1956 in the amount $2,500,000 to yield 2.60 per cent. The maximum bond interest authorized to be paid by the Authority is 5 per cent. The largest rate presently being paid is 4 per cent on the Series "C" issue sold June 1, 1936 with final maturity on June 1, 1964. The lowest interest being paid is 1.875 per cent on the Series "J" issue sold March 15, 1949 with final maturity on March 15, 1966. The final maturity date for currently outstanding bonds is June 1, 1980 for the Series "O" issue.

Bonds of the Authority are currently rated Aa by Moody's Investor Service. Bonds having this rating are judged to be of high quality and together with the Aaa group comprise what are generally known as high grade bonds. Bonds of the City of Buffalo and County of Erie are rated Aaa.

Commencing with the Series "I" issue, bonds of the Authority have contained a call provision, generally not operative for the first six years, at a

decreasing premium for the next 10 years, and then at par for the remaining life of the bond. This provision was embodied principally to make possible the extinguishment of the Authority if desired by the City upon meeting all the Authority's liabilities and paying in full all of its bonds. The last non-callable bonds will mature on November 15, 1967. The call feature also has the obvious advantage of refunding high interest bearing bonds during cheap money periods. The Authority has so far not engaged in any refunding operation.

The Authority is fortunate in that there are no restrictions in its scheduling of bond maturities. It has thus been possible to defer maturities of newly sold bonds beyond the heavy maturity schedule of the early bond issues, based of course on the obvious desirability of keeping annual debt service costs as near uniform in amount as is possible or at least obviating creation of unwieldy variations in annual revenue requirements to support debt service. The Authority attempts to plan its construction program in harmony with a fiscal program based upon minimum fluctuations in revenue requirements.

The Authority's bond indentures provide for the establishment of a Bonds Payable Reserve Fund and a General Reserve Fund. Monthly payments are made to the Bonds Payable Reserve Fund to meet current payments of principal and interest and in additional amounts to provide a reserve to safeguard future such payments. These additional amounts equal one-fifth of the monies set aside for the principal and interest payments and such reservations are made until the accumulated amounts, identified for convenience as the "1/5 Reserve Fund," equals the interest and principal due on the next four succeeding interest dates. Presently this 1/5 Reserve Fund contains $1,800,000 to safeguard the next two principal payments and next four interest payments for $12,295,000 in outstanding bonds.

The General Reserve Fund is intended to serve in case of any financial emergency and at such time provide for the payment of operating expenses and interest and principal of bonds. This Fund must be kept at least equal to 2-1/2 per cent of the principal amount of all authorized bonds and presently contains $463,500.

The Authority keeps its reserve fund monies invested to the fullest extent in United States Government securities. The 1/5 Reserve monies have been invested in Series G, Series K and medium term Treasury bonds, giving consideration to the times when such reserve money will be needed for final maturity payments of certain issues. The General Reserve monies have been invested in short-term Government securities in preference to longer term issues on the general premise that such monies should be quickly available, without serious marketing loss, in event of any emergency requiring the same.

Monies obtained for construction and improvement purposes from the sale of Authority bonds are invested in United States Government short-term securities until needed. The most adaptable security for this purpose appears to be the 91-day Treasury Bills now yielding in the neighborhood of 3 per cent. These bills are providing a profit to the Authority on the unused balance of its Series "O" bond money for which it is paying 2.6 per cent. Idle operating funds of the Authority and idle bonds payable funds are similarly invested.

The Authority attempts to maintain a modest surplus account in the neighborhood of $300,000.00. This account is deemed desirable to make up for any unexpected appreciable drop in revenues, to provide a source of moneys for any emergent need or unforeseen major expense, and to provide a source of revenue for budgetary purposes thereby somewhat stabilizing sewer rental

rates. It appears to particularly merit its existence at times of bond sales when, in the investor's view, it serves to improve the generally attractive financial pattern of the Authority.

The Authority has a "Capital Improvement Fund" for financing minor improvements on a pay-as-you-go basis. The Fund is an independent accounting entity on the books of the Authority and is supported by appropriations from the operating budget of the Authority of not less than 2 per cent of the total appropriation of the budget for the fiscal year. Expenditures from the Fund are limited to specific improvements not exceeding $50,000 in cost and the total unexpended balance in the Fund is not to exceed $300,000. Because of continuing operating budgetary difficulties in this period of constantly increasing costs, annual appropriations to this Fund have frequently been waived and it has accordingly not been as useful as originally intended.

The paramount cause for capital expenditures by the Authority is the great need in the City for storm water relief facilities. The combined sewer system of the City inherited by the Authority in 1938 had its beginnings over 100 years previous and many sewers are in service today that are over 100 years old. No data exists on the method of design for sewers constructed prior to this century. Sewers constructed between about 1900 and 1935 were in general designed to handle storms of 5 year frequency.

Studies by the Authority's engineers in the latter 1930's resulted in a recommendation that the existing system be reinforced by storm water relief drains to enable it to care for storms of 10 year frequency. Although many such drains were constructed under WPA auspices prior to the war and by contract since the war, there remains over $60,000,000 in drains to be constructed before the above objective will be fully attained.

The accomplishment of such a program by the Authority presents aspects of impracticability if not impossibility for the foreseeable future. The Authority has, therefore, had to appraise this matter in terms of its financial capabilities as measured by the indebtedness limitation imposed by the legislative act as well as by public reaction to continually increased rental charges.

The Authority, therefore, adopted a construction priority schedule which would merit general public acceptance and at the same time harmonize with what it conceives to be a sound financing program. From the list of needed storm water relief drains estimated to cost upward of $60,000,000 were selected projects aggregating $13,000,000 in cost. The basis of selection was on the principle that highest priority ratings be given to those projects providing the greatest benefits per dollar invested.

To measure the benefit per dollar of cost, a formula was developed to take into account all tangible factors of influence. The formula includes a factor for the area to be relieved both directly and indirectly by the project, a factor for the relative assessed valuation, i.e. the ratio of average assessed valuation of the area benefited to the average assessed valuation of the entire City, a factor for the degree of development, i.e. the ratio of present population to ultimate expected population of the district served, a factor for the average percentage of adequacy of the existing sewers serving the area, and a factor for the estimated cost of the relief project, including branches, laterals and inlets. Thus the priority rating is high for areas of high value and advanced development and is influenced by acreage, adequacy of existing drainage facilities and cost of providing required relief. Like every formula of this character, it must to some extent be qualified by engineering experience and judgment.

Storm water relief projects are paid for in their entirety by the Authority without resort to local assessment or direct property charge. Sanitary or combined sewers to serve hitherto unsewered or undeveloped areas are paid for by and are a direct charge to the property benefited. The Authority does not have the right of assessment. It, therefore, handles the matter of payments for sanitary or combined sewers on the basis of prior agreement with the owners of properties benefited. Such agreements may provide for payment prior to construction or for payment upon making connection. The Authority thus helps to finance the construction of some sewers and recovers later when the property is developed and a connection made. Occasionally the Authority permits the developer of property to construct needed sewers from plans and specifications prepared by the Authority and under Authority inspection. Such sewers are accepted by the Authority as public sewers when satisfactorily completed.

Whereas municipalities frequently are self-insured, and this observation is particularly applicable to the City of Buffalo, the Buffalo Sewer Authority has since its beginning purchased coverage with established insurance companies. This requirement has also been incorporated in the bondholders' agreements to the end that the Authority will maintain insurance on its facilities adequate in amount as to risks involved. The coverage has been on (1) loss of or damage to its own properties and (2) legal liability for loss of or damage to property owned by others or for injury of others by reason of the Authority's operations. It is the policy of the Authority to name the City of Buffalo as an additional insured.

Policies carried against loss of or damage to Authority properties, are as follows:

1. On buildings and contents --- all direct loss and damage by fire including perils of nature and civil activities, vandalism and malicious mischief.

2. On automotive equipment --- comprehensive coverage against fire, theft, windstorm, etc.

3. On equipment not regularly stored or housed but generally on location where required in performance of work --- direct loss or damage caused by fire, collision, malicious mischief, vandalism, theft, etc.

Policies carried against loss by reason of legal liability incurred in conducting the business of the Authority are as follows:

1. Compensation coverage is required by law indemnifying employees of the Authority (including members of the Board) by reason of injury or death (including occupational disease) while engaged in operations of the Authority.

   (Note: It has been the Authority's policy when injuries are sustained by its employees to continue such employees at full pay during the first 30-days of their disability. When and if compensation payments are made for that period, the Authority is reimbursed in such amounts.)

2. On automotive equipment including operation of any non-owned automobile on Authority business --- for bodily injury and property damage.

3. On property of the Authority and others for damage resulting from boiler explosion.

4. On all operations of the Authority --- comprehensive liability coverage with limits of $100/300,000 for bodily injury and $50/100,000 for property damage --- including:

   a) All premises owned or occupied by the Authority.

   b) Direct operations or work performed by the Authority's employees.

   c) Existence hazard as represented by 776 miles of sewers, 15,000 manholes, 20,000 stress receivers, and various other appurtenances.

   d) All work performed by independent contractors, either direct contract or subcontract.

   e) Liabilities arising out of easement agreements.

In New York State the concept of depreciation has not gained particularly wide acceptance in public authority accounting. Authorities have generally tended to follow the example of the State which does not depreciate its capital facilities. The Buffalo Sewer Authority has as a result not depreciated the value of its capital assets except for insurance purposes. Effort is made to keep all facilities in constant repair for full use. So far as possible, maintenance, repair and replacement of capital facilities depend on annual appropriations. Replacement of capital facilities has not constituted a financial burden, even with portions of the sewer system over a century old.

Although the City Comptroller has been empowered to audit from time to time the accounts and books of the Authority, the Authority has regularly had a semi-annual post-audit performed by an independent auditor, whose audit has been accepted by the Comptroller.

The Authority issues a complete report annually covering its operations and in particular its fiscal affairs. It is required that a copy of this report be submitted to the Governor and to the State Comptroller. It has been customary to also send a copy to the Clerk of the State Senate and to the Clerk of the State Assembly. The fiscal section of the report is prepared to conform with the recommendations of the Municipal Finance Officers Association and the National Committee on Governmental Accounting. The Authority was awarded a certificate on such conformance in 1950. The last issued report was awarded the William D. Hatfield Award of the Federation of Sewage and Industrial Associations.

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FINANCIAL STATISTICS
BUFFALO SEWER AUTHORITY
1955-56

Revenues

Sewer Rent based on Assessed Valuation of Real Property	$1,347,000.00	55.1%
Sewer Rent based on Water Consumption	976,000.00	39.9%
Sewer Rent - Miscellaneous	54,000.00	2.2%
Interest Income	61,000.00	2.5%
Other Income	7,000.00	0.3%
	$2,445,000.00	100.0%

Expenditures

Debt Service	Principal	$555,000	23.3%
	Interest	320,000	13.5%
	Reserves	75,000	3.1%
	Miscellaneous	4,000	0.2%
		$954,000	40.1%
Sewage Disposal	Pumping	$145,000	6.1%
	Gen. Treatment	260,000	10.9%
	Digestion	135,000	5.7%
	Filtration	100,000	4.2%
	Incineration	140,000	5.9%
	Pension	38,000	1.6%
	Insurance	24,000	1.0%
	Miscellaneous	25,000	1.1%
		$867,000	36.5%
Sewer System	Engineering & Construction	$110,000	4.6%
	Maintenance & Repair	270,000	11.3%
	Pension	21,000	0.9%
	Insurance	19,000	0.8%
		$420,000	17.6%

Expenditures (continued)

Administration	General	$130,000	5.5%
	Pension	5,000	0.2%
	Insurance & Miscellaneous	2,000	0.1%
		$137,000	5.8%
Total		$2,378,000	100 %

Sewage Disposal Expenditures for Salaries, Materials, Power, etc.

Salaries	$549,000	63.2%
Pension	38,000	4.4%
Materials & Supplies	142,000	16.4%
Power and Light	57,000	6.6%
Maintenance-Materials	25,000	2.9%
Insurance	24,000	2.8%
Bridge Operation	20,000	2.3%
Water	3,500	0.4%
Miscellaneous	8,500	1.0%
	$867,000	100.0%

1955-56 Unit Costs

Total Treatment cost	$14.94 per M. G.
Pumping	2.77 "
General Treatment	5.01 "
Digestion	2.58 "
Filtration	1.86 "
Incineration	2.72 "
Dry solids removed	
Pumping	$15.21 per ton
General treatment	27.48 "
Dry sludge solids	
Pumping	19.27 per ton
General Treatment	34.81 "
Digestion	17.93 "

Sludge cake removed	
Filtration	5.13 per ton
Sludge cake burned	
Incineration	7.92 per ton
Dry solids burned	
Incineration	19.60 per ton
Sewer Cleaning	14.9 cents per lin. ft.
Receiver Cleaning	80.2 " " receiver
Catch-basins Cleaning	$1.40 per catch basin

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Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

EFFECT OF LOCAL WEATHER ON AIR-POLLUTION PROBLEMS[1]

A. L. Danis[2]
(Proc. Paper 1463)

SYNOPSIS

Valid findings in the study of an air-pollution problem at a given site can be made only from local meteorological data. Wide variations in data between two adjacent Florida localities are cited. Five typical cases showing the effect of local weather on the diffusion of air-borne pollutants are discussed.

A study of climatic data is a prerequisite to site selection, planning, and plant design for an industry having an air-borne waste disposal problem. The data should provide information useful for stack gas diffusion estimates, engineering, and construction needs. This paper is concerned primarily with diffusion climatology.

The ability of the atmosphere to disperse and diffuse a polluted effluent from any source varies widely with weather conditions. It is as much as a thousand times greater under certain circumstances than others.

The effects of this great variability in the characteristics of weather must be studied and analyzed. At any site, the probable percentage of time of good diffusion or dispersion of air-borne pollutants should be known. The remaining number of hours with poor diffusion characteristics of the atmosphere then becomes a problem for the engineers and plant management. It is this latter period which is of greatest concern.

Meteorological conditions which affect air pollution problems can be considered under two headings—climate and weather.

Climate is the average condition of weather at a place over a period of years. It varies widely from one portion of the continent to another. So the general climatic conditions of a site should be known before a close study of local weather is made. During any given season, the properties of the

Note: Discussion open until May 1, 1958. Paper 1463 is part of the copyrighted Journal of the Sanitary Engineering Division of the American Society of Civil Engineers, Vol. 83, No. SA 6, December, 1957.

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atmosphere vary greatly, depending on whether the site is in the mountains, along the coast, or on the plains. In one area the winds may be generally light and the diffusion of smoke and gases slow, while in another area the opposite may be true with predominantly strong and variable winds associated with rapid diffusion.

The rate of diffusion is governed mainly by the degree of atmospheric turbulence present. Turbulence is in itself an extremely complex phenomenon and varies with weather conditions. It depends on such phenomena as wind velocity, rate of change of wind speed, wind direction, and change of temperature with height. Since these conditions differ widely by season and by location, there is a corresponding variation in the diffusing capacity of the atmosphere. After this general information is available, an estimate of the difficulty or complexity of the air pollution problem can be made and more intelligent use of local weather conditions will result.

In the study of local weather at a site, attention should be directed toward causes of local variations in weather. In general, it is little realized how great these variations can be. Local differences are dependent on topography; soil constitution; degree, type, and location of vegetation cover; amount of free surface water; location of built-up areas; and similar factors. They are so complex as to make the prognosis of atmospheric conditions futile unless based on local meteorological observations.

It is easy to see the effect on air flow over rough terrain such as mountainous country where valley winds, as well as eddies developing along the mountain sides, would be expected. Again over flat country, one would normally observe more of a gradient flow of air during the day. At night, because of thermal stratifications, the density of the air changes near the ground and damps out the eddies or turbulence. The effect of surface friction is thereby increased and, in the case of light to moderate winds, holds the lower air comparatively quiet. The air flow at night in the lower levels of the atmosphere then tends to follow the ground contours, flowing like a meandering water stream through flat country. The rate at which the temperature of the air changes is closely associated with the type of soil, vegetation cover, and water surfaces in the surrounding area. It is known that heat will radiate more rapidly from a sandy soil than black loam, from a grass field than a forest, and from land than from water. This varying interchange of heat between the ground surface and the atmosphere modifies the temperature of the free air mass flowing over the ground and thereby affects its density. It is this effect which gives the atmosphere either an unstable or stable equilibrium. In this study of the dispersion of contaminants in the atmosphere, the problem resolves itself into one of determining when the atmosphere at the lower level develops a stable equilibrium.

In order to analyze this state of the atmosphere, as well as the areas to be affected by any air-borne pollutant, the local meteorological conditions must be known. The atmospheric phenomena which have a direct bearing on the diffusion or dispersal of gases are pressure, wind, temperature, state of weather, and humidity. These phenomena are all familiar but each takes its part in this problem.

It might be appropriate to indicate first these equilibrium conditions in the atmosphere by noting Figure 1 showing various temperature profiles. In general, the air becomes cooler with an increase of altitude. The normal or standard lapse rate shows a temperature decrease of 3.5°F per 1000 feet. This takes into consideration the presence of water vapor in the air. For dry

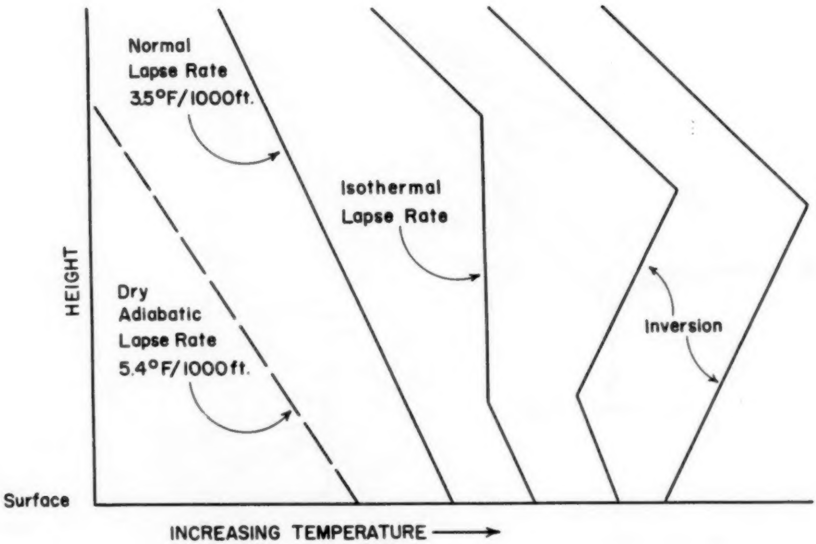


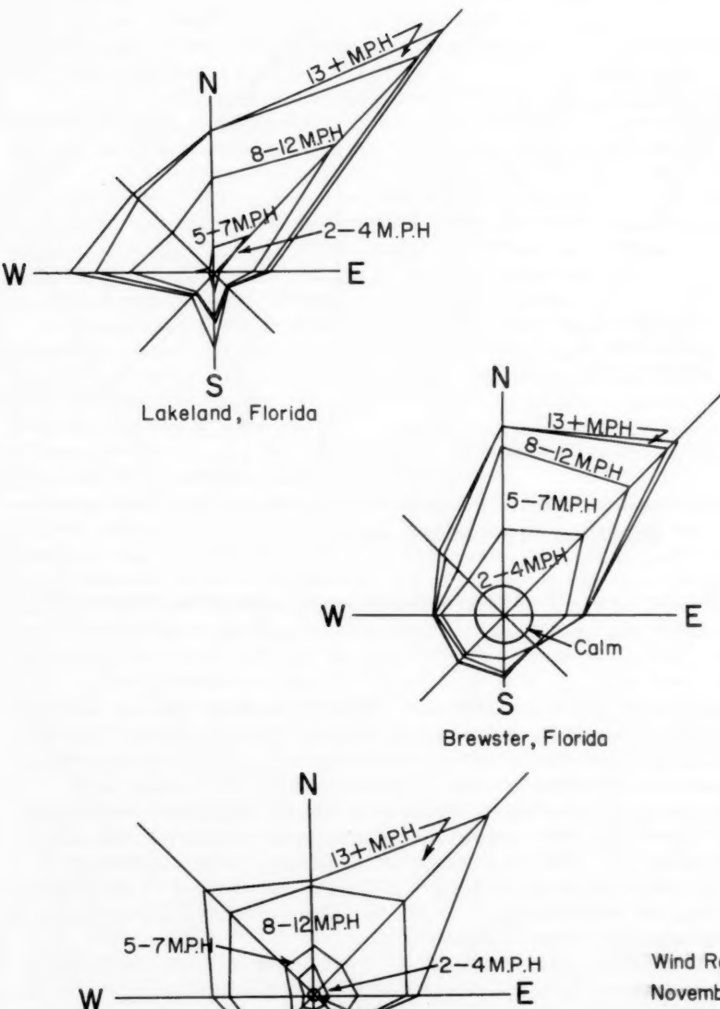
FIGURE 1. Temperature Profiles illustrating various Lapse Rates which occur in the atmosphere.

air, the lapse rate is 5.4°F per 1000 feet. This is known as the dry adiabatic lapse rate (DALR), and if the lapse rate is equal to this or greater, then the atmosphere is unstable. When the lapse rate lies between the dry adiabatic lapse rate and an isothermal condition, which is one in which there is no change in temperature with height, the atmosphere is considered conditionally unstable. Its stability will depend on the amount of moisture in the atmosphere. A stratum which has an increase of temperature with altitude is recognized as having an inverted gradient. Such a phenomenon is designated as an inversion and the atmosphere is stable. The stronger the inversion, the greater is the stability of that stratum of air.

As previously stated, wind direction, air speed, and conditions of equilibrium of the atmosphere have a direct bearing on any air-pollution problem. Because of this fact, local meteorological data are most important. Data on weather 25 to 50 miles away from a site cannot be the basis for valid findings. For instance, wind roses were constructed for November, 1956, for Lakeland, Brewster, and Gainesville, Florida. Lakeland is in the central part of Florida and was used as a reference base for the other two stations. At Lakeland there is a first order U. S. Weather Bureau station which has reliable weather records over a long period. It is on the ridge of land running through central Florida.

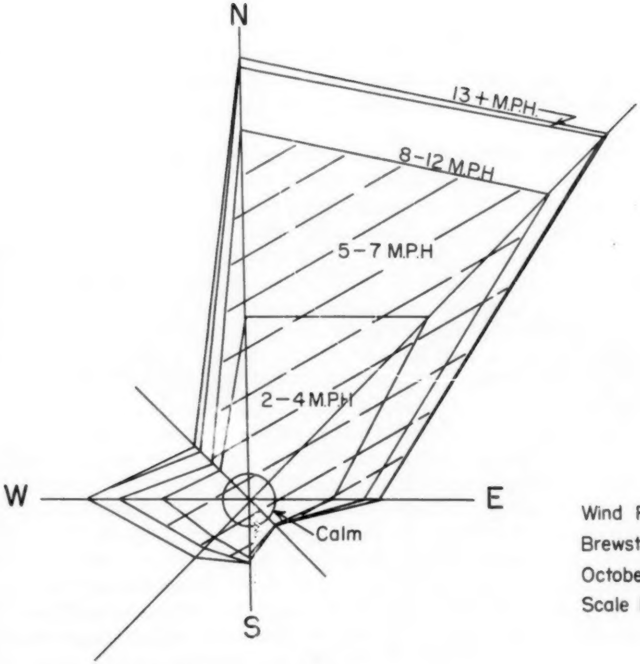
Brewster is 22 air miles south of Lakeland. It lies in the flatland between the ridge and the west coast of Florida.

Gainesville is in the north-central part of the state, 125 miles from Lakeland.



Wind Roses
November, 1956
Scale 1cm=40hrs

FIGURE 2



Wind Rose
Brewster, Florida
October, 1956
Scale 1cm = 20 hrs

FIGURE 3 (a)

Modified Wind Rose
Brewster, Florida
October, 1956
~~Scale 1cm = 20 hrs~~

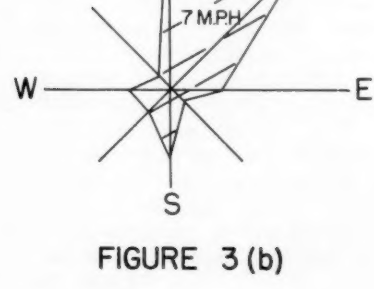


FIGURE 3 (b)

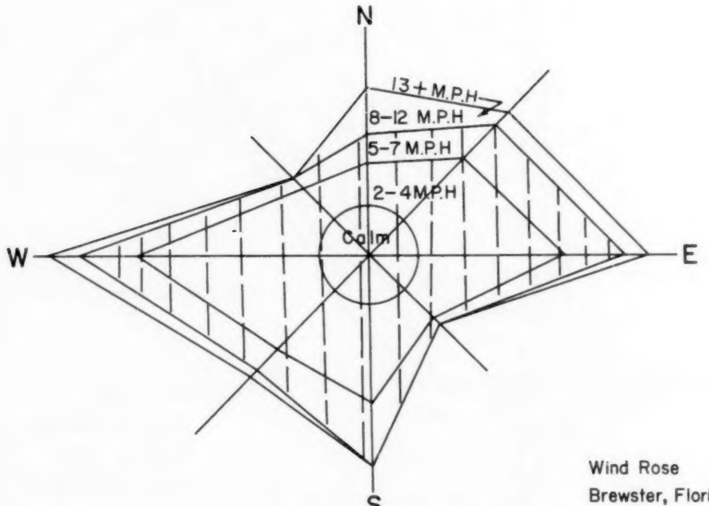


FIGURE 4 (a)

Wind Rose
Brewster, Florida
January, 1957
Scale 1 cm = 20 hrs

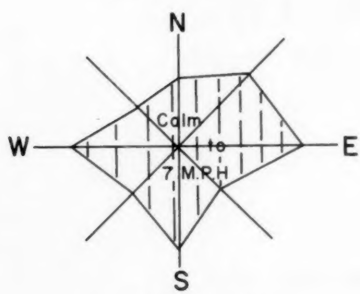


FIGURE 4 (b)

Modified Wind Rose
Brewster, Florida
January, 1957
Scale 1 cm = 20 hrs

Note in Figure 2 both the variations in wind direction as well as the number of hours of various wind speeds during the month. Although Brewster and Lakeland are only 22 air miles apart, the topography causes the wind to back more to the north at Brewster and its velocity decreases. The example illustrated in Figure 2 clearly shows how meteorological data for Lakeland is of no more use in solving problems in air pollution at Brewster than at Gainesville.

Winds over 7 mph usually will create such turbulence that the atmosphere in the lower levels will remain unstable. Therefore, there are left two weather phenomena to consider and analyze. One is the number of hours in which there are winds of 7 mph and below, and their hourly direction. The other is the changing stability of the atmosphere and the duration of a conditionally unstable or stable equilibrium. Although winds of 7 mph and below are favorable for ground fumigation, it also requires certain conditions of equilibrium in the atmosphere to bring any contaminant to the surface, for frequently smoke is seen to rise vertically even on a calm day.

Because of this observed phenomenon, the following analysis of weather at Brewster was made for the months of October, 1956, and January, 1957. The hourly temperature profiles were analyzed for the time of the beginning and ending of stable conditions. For these daily periods, a second wind rose was constructed for winds under 7 mph. In October winds of 7 mph and below occurred at Brewster for 504 hours while in January they occurred for 653 hours. On completion of the analysis and after considering the periods during which the air was in stable equilibrium, the probability of ground fumigation was reduced from 504 to 248 hours in October and during January, from 653 to 326 hours. This condition also modified the sectors in which maximum fumigation might be expected, as well as the number of hours it might be expected to persist in those areas.

Figure 3(a) shows the October, 1956, wind rose for Brewster and Figure 3(b) shows the modified wind rose for the hours during the same period in which heavy ground fumigation could occur.

Figures 4(a) and (b) give similar information for January, 1957.

Considerable study has been made of the behavior of the effluent plume from a stack under different conditions of stability in the atmosphere. The extent of the area affected would depend on the wind speed. In the following five[1] cases, the diffusion characteristics of an effluent from a point source under these varying conditions of stability are discussed.

Case 1, Figure 5, shows an unstable condition in which the stack effluent seems to roll and twist because of the thermal eddies in the air flow. Gases diffuse rapidly, but there is a slight possibility of pollutants being brought to ground level in the vicinity of the base of the stack. The lower the wind velocity the more nearly vertical will the plume rise. This condition in the atmosphere should not cause too much worry in plant operations.

Case 2, Figure 6, shows a weak or conditionally unstable lapse rate which gives a coning effect to the effluent. In this type of gradient, the vertical motion is less and, therefore, if any pollutant reaches the ground it will be at a greater distance from the stack than in the previous case. Strong fumigation at the surface should not be expected under these conditions except that resulting from terrain configuration or similar causes.

Case 3, Figure 7, shows how a continuous inversion condition will cause the effluent to fan out with very little vertical diffusion. If winds are light, the plume may meander over flat terrain. The contaminated plume from a

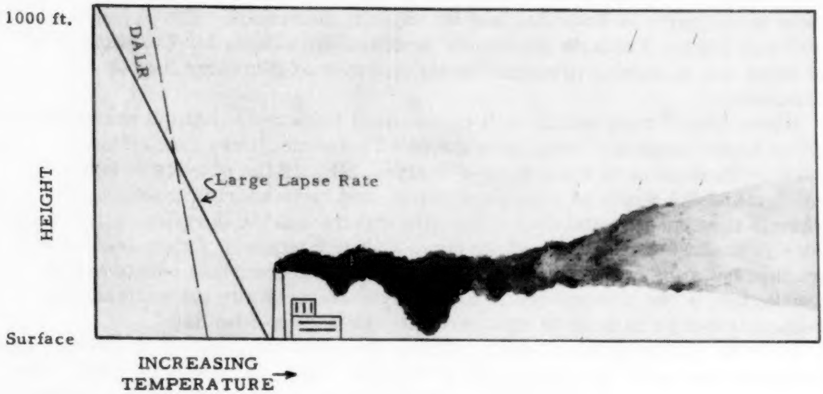


Fig. 5.

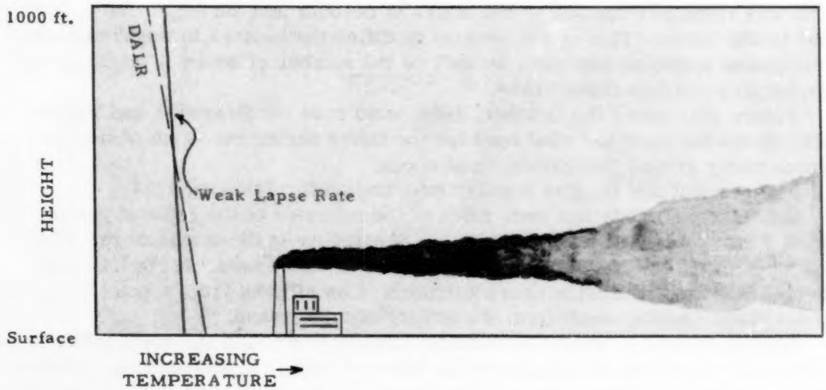


Fig. 6

point source will widen and spread and the concentration of the pollutants in the stream may vary to a considerable extent over the area in question. As long as the inversion lasts, ground level fumigation is very unlikely.

Case 4, Figure 8, shows what happens when an inversion exists below the stack height with a normal lapse rate above. This will give the most favorable diffusion conditions. This occurs generally at sunset or in early evening, and the length of time during which it is favorable for maximum pollutant diffusion is based on other meteorological conditions. The inversion in this case acts as a lid to protect the ground surface from fumigation.

At some sites, an inversion may develop during the night hours with its

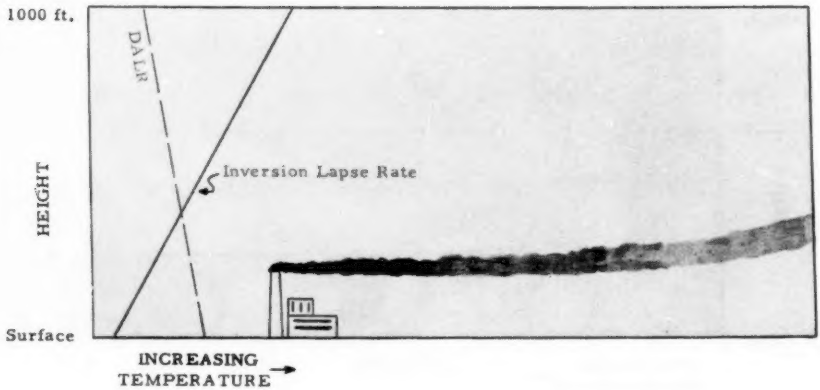


Fig. 7.

base well above stack height. If, in the morning, the inversion breaks from above, all the pollutants trapped in the inversion during the night hours will be diffused to the upper atmosphere. On the other hand, if the nocturnal inversion is dissipated by the sun heating the ground, which is usually the case, then conditions exist for maximum fumigation as noted in the next case.

Case 5, Figure 9, where the night inversion is dissipated by insolation, an adiabatic lapse rate begins at ground level and gradually forces the base of the inversion upward. As it reaches the level at which the pollutants have been flowing with little diffusion and possibly some concentration, convective eddies are set up, which mix the contaminated stratum of air with the shallow

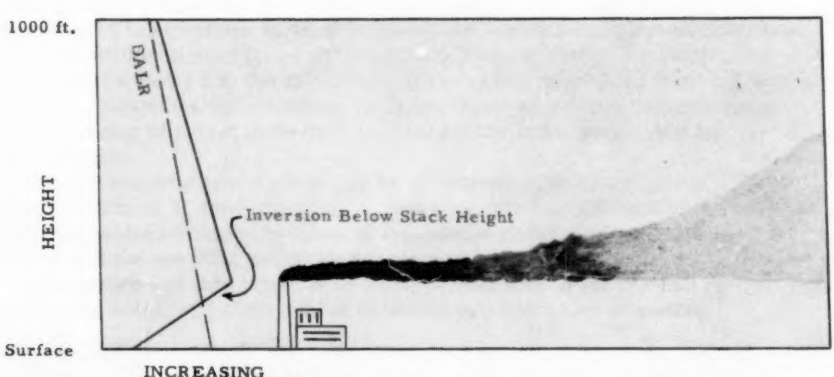


Fig. 8

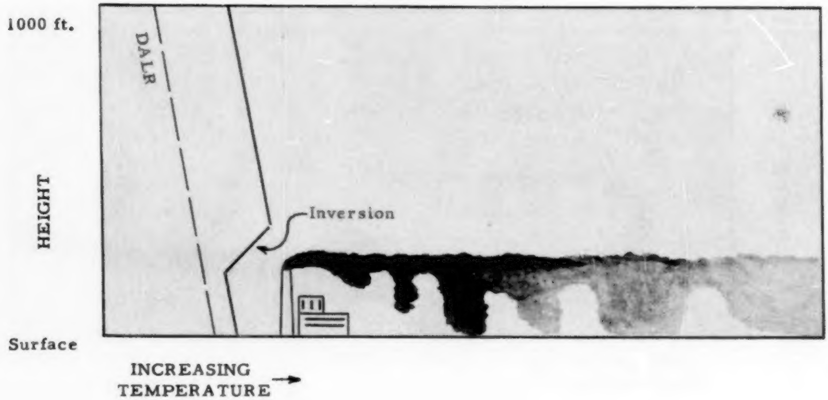


Fig. 9.

layer of air between it and the ground. Large concentrations of pollutants are brought to the ground surface in this manner. This is one of the worst conditions for ground level fumigation.

From this short discussion of some of the physical properties of the atmosphere, one can visualize the importance of local meteorological conditions on the evaluation of atmospheric pollution problems.

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Paper 1464 December, 1957 SA 6

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SODIUM METAPHOSPHATE GLASS IN WATER TREATMENT

David H. Howells,[1] A.M. ASCE
(Proc. Paper 1464)

SYNOPSIS

Sodium metaphosphate glass can be a useful tool in municipal water treatment if properly applied. This paper attempts to provide, through a review of the literature and field practices, a background of information regarding sodium metaphosphate glass requisite to its effective utilization by the sanitary engineer.

INTRODUCTION

The sanitary engineer practicing in the field of municipal water supply must have a competent grasp of water chemistry if his designs are to provide for maximum efficiency and economy in the treatment of waters of varying quality.

Of the constantly growing list of chemicals used in municipal water treatment, probably none has been less understood and more universally misrepresented than sodium metaphosphate glass. This is understandable, due to the multitude of conflicting articles purporting to define its application which have appeared in the literature, the complex chemistry involved, occasional misapplication resulting from overzealous selling, and ambiguity due to the use of patent protected product names and erroneous chamical nomenclature.

Sodium metaphosphate glass can be of considerable value in municipal water treatment if used effectively. However, the full efficacy of its use has been too seldom attained because of inadequate knowledge as to its chemical nature and the specific limitations of its use.

The purpose of this paper is to make information available to sanitary engineers which will enable them to obtain maximum use of sodium

Note: Discussion open until May 1, 1958. Paper 1464 is part of the copyrighted Journal of the Sanitary Engineering Division of the American Society of Civil Engineers, Vol. 83, No. SA 6, December, 1957.

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metaphosphate glass in the treatment of municipal water supplies. This information was obtained through an exhaustive search of the literature, discussions with research chemists and sales engineers concerned with the development and sales of this and other condensed phosphates, and through personal contacts with water works operators.

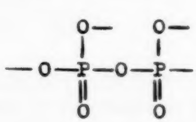
Chemistry of Sodium Metaphosphate Glass

Chemical Structure

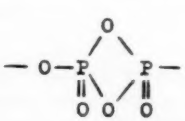
The phosphates as a chemical family include the salts of orthophosphoric acid, and a secondary class of phosphate compounds, the members of which are formed by the molecular dehydration of the orthophosphates. This latter group is known collectively as the condensed or molecularly dehydrated phosphates.

The anions of the phosphates contain a phosphorus atom surrounded by four oxygen atoms situated at the corners of a tetrahedron. Single tetrahedra are orthophosphates. The condensed phosphate anions are composed of more complex structures of combined tetrahedra in the form of chains, rings and branched polymers produced by the sharing of oxygen atoms between tetrahedra.[1] Bond angle requirements restrict the sharing of oxygen atoms to no more than two and probably only one oxygen atom between any two tetrahedra.[2]

Single Connection



Double Connection



All phosphates can be represented chemically as combinations of oxides in terms of a metal oxide, water of constitution, and P_2O_5. For example, disodium orthophosphate and sodium tripolyphosphate can be shown as follows:

Empirical Formula		Resolved Formula
2 Na_2HPO_4	or	2 $Na_2O \cdot H_2O \cdot P_2O_5$
2 $Na_5P_3O_{10}$	or	5 $Na_2O \cdot 3P_2O_5$

The molar ratio of cationic oxides such as Na_2O, CaO, and including H_2O of constitution to the anionic oxide P_2O_5 will determine the type of phosphate.

If the molar ratio of the cationic to anionic oxides: $\left(\frac{Na_2O + H_2O + ---}{P_2O_5} \right)$ is three, the compound is an orthophosphate. Phosphates with a molar ratio of less than three are classified as condensed or molecularly dehydrated phosphates. Table I illustrates the effect of changing oxide ratio upon phosphate structure and classification.

Table I

Classification of Phosphates

Oxide Ratio	Common Chemical Name	General Formula of Normal Salt	Structure
		Condensed Phosphates	
0	Phosphorus pentoxide	$(P_2O_5)n$	P_4O_{10} molecules or continuous structures
0-1	Ultraphosphate	$(xM_2O)P_2O_5$	Interconnected chains or rings
1	Metaphosphate	$Mn(PO_3)_n$ $n=3,4,- -$	Rings or extremely long chains
1-2	Polyphosphate	$M_{n+2}P_nO_{3n+1}$ $n=2,3,4,5,- -$	Chains
2	Pyrophosphate	$M_4P_2O_7$	Two phosphorus atoms
2-3	Mixtures of pyrophosphate and orthophosphate	- - - - - -	- - - - - - - - - - - - -
		Orthophosphates	
3	Orthophosphate	M_3PO_4	One phosphorus atom
>3	Orthophosphate metal oxide	- - - - - -	- - - - - - - - - - - - -

Adapted from table by Encyclopedia of Chemical Technology[1]
M = any monovalent metal

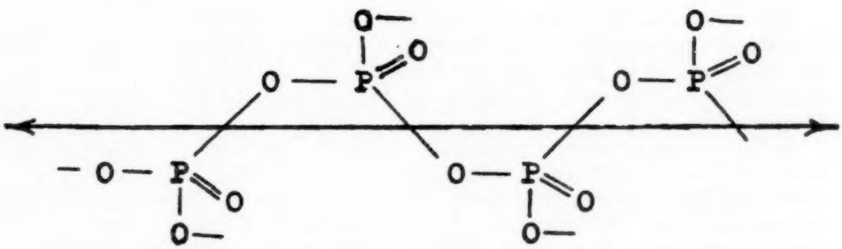
The phosphate formula, $M_{n+2}P_nO_{3n+1}$, in which M represents any univalent metal and n any assigned interger, is of interest when considering the ortho- and condensed phosphates. When n = 0, the formula becomes that of the metal oxide; when 1, orthophosphate; when 2, pyrophosphate, and when 3, tripolyphosphate. As n becomes very large, the formula becomes analytically indistinguishable from that of the metaphosphate. However, none of the theoretical members of this formula series other than ortho-, pyro-, tri-, and metaphosphate has been found to exist in crystalline form.(3)

A very large number of the condensed phosphates are noncrystalline. Among the compounds of this class the most interesting are the sodium phosphate glasses which can be made over a wide range of composition extending from pure P_2O_5 to a Na_2O/P_2O_5 molar ratio of 1.7. This has resulted in a countless number of possible compositions which cannot be accurately identified by the usual chemical terminology. As an example, the metaphosphate glass of molar ratio of 1.0 is known as "sodium hexametaphosphate." This title carries the incorrect implication of six $NaPO_3$ units linked together. Consequently, it has been suggested that the molar ratio of Na_2O to P_2O_5 of 1.0 be used to define this compound. Due to the presence of impurities such as H_2O, CaO, MgO, and SO_3, in commercial glasses a clear-cut method of description might be to use a statement such as "sodium phosphate glass containing n percent of P_2O_5."(3) Compounds presently being sold as "sodium hexametaphosphate" contain 67 percent and less P_2O_5 as compared to the theoretical 69.6 percent P_2O_5 content of a pure Na_2O-P_2O_5 glass of unity molar ratio. The term "sodium metaphosphate glass" will be used throughout the remainder of this paper to designate the sodium phosphate glasses of Na_2O to P_2O_5 ratio of approximately 1 which are in common use today throughout the water supply industry and are identified by trade name or as "sodium hexametaphosphate."

If a pure anhydrous sodium metaphosphate glass for which the Na_2O to P_2O_5 ratio is equal to or greater than unity consists of only chain phosphates with no rings or orthophosphate present the average chain length, $\bar{n}$ can be obtained by: $\frac{Na_2O}{P_2O_5} = \frac{\bar{n}+2}{\bar{n}}$ From this equation, it can be seen that under the stated conditions the chain length would be infinite with a Na_2O to P_2O_5 ratio of exactly one. In practice, however, it is difficult to volatilize the last trace of water from the chemical and this water of constitution, together with other impurities, prevents the formation of chain lengths in excess of about 250 phosphorus atoms.(4)

Substantial evidence has been offered to support the belief that rod-type molecule-ions predominate in solutions as Na_2O-P_2O_5 glasses, and that these ions are orientated. In salt-free solutions it is reasonable to expect that the numerous negative charges on the polyelectrolyte chain would cause it to extend to its most elongated orientation as shown below:(5)

On the basis of conductivity measurements of a solution of sodium metaphosphate glass, Davies, et al,(6) concluded that the anion has an extremely high molecular weight. Data were produced to show that the high molecular weight of the molecule-ion is not due to loose association of the numerous units of low molecular weight to form a micelle.



Chemical Properties

The condensed sodium phosphates possess a number of different properties which make them extremely useful in water treatment. Three of the chain homologs, tetrasodium pyrophosphate, sodium tripolyphosphate and sodium metaphosphate glass have found the greatest use in this respect.

These condensed phosphates have the ability to form soluble complexes with metal cations. The amount of dissociation of the complex will depend upon the particular metal ion involved. For instance, the alkali metals such as sodium and similar cations are weakly complexed. Strong complexes are formed with multivalent metal ions such as iron, manganese, aluminum, calcium and magnesium. The complexes have not yet been defined in terms of a chemical formula and dissociation constants. However, present indications are that the ability of the condensed phosphates to form complexes is independent of their chain length, is proportional only to the total number of phosphorus atoms in the molecule ion, and is dependent upon the absence of spatial restriction of the P - O - P connections by small rings.(1,7)

The sequestering capacity of the condensed phosphates is markedly affected by changes in pH. At 54.5°C the amount of a sodium metaphosphate glass (65 percent P_2O_5) required for calcium sequestration has been found to increase three-fold as the pH of the solution increased from 6.7 to 9.1. This effect decreases as the pyrophosphate P_2O_5 content is approached with pyrophosphate itself not being much effected.(8)

Sodium metaphosphate glass and certain other of the condensed phosphates act as a deterrent to the crystallization of calcium carbonate in very low or "threshold" concentrations. Such concentrations are far below those required for calcium suppression through sequestration, yet the two phenomena are probably related. The inhibiting action is presumably due to adsorption of the condensed phosphate molecule ions on the submicroscopic nuclei resulting in gross distortion of the calcite crystals and preventing normal crystal development. This results in the stabilization of an otherwise unstable, supersaturated solution of calcium carbonate.(8)

The molecule ions of sodium metaphosphate glass are also strongly adsorbed on suspended particle surfaces and cause deflocculation through the resulting increase in surface potential. When all of the particles have developed strong charges of the same sign, they repel each other and are deflocculated. In addition to its surface active properties, sodium metaphosphate glass has been found to exhibit the cation exchange properties of a

typical polyelectrolyte.

Hydrolytic Degradation

The condensed phosphates hydrolyze in aqueous solution and ultimately give the orthophosphate form. The rate of hydrolysis for any particular condensed phosphate is dependent upon temperature, pH, concentration of phosphate, ionic environment, and biological enzyme systems which may be present.(10)

Hydrolysis is a stepwise process in which the condensed phosphate structures become less and less condensed until finally nothing is left but the uncondensed form, the orthophosphate. For long chains of alternate phosphorus and oxygen atoms, the hydrolytic cleavage of P-O-P- linkages along the chain appears to be completely random and the rate of linkage splitting is independent of molecular size until the pyrophosphate is reached.(1)

Hydrolysis of the condensed phosphates is greatly accelerated by increasing temperature and is a relatively slow process at room temperature and neutral pH.(1,11)

Crowther, et al,(12) measured the rates of hydrolysis of sodium pyrophosphate and sodium tripolyphosphate at 65.5°C over a pH range of 2 to 12 and found that the hydrolytic reactions were first order in nature. Both reactions were found to be acid catalyzed, but only the hydrolysis of the tripolyphosphate was found to be base catalyzed. Bell(11) reported base catalysis in the hydrolysis of sodium metaphosphate glass and results similar to those of Crowther for sodium pyrophosphate and tripolyphosphate.

The presence of calcium or other cations which the condensed phosphates can complex is believed to effect the rate of hydrolysis. This rate is known to be increased in the presence of colloidally precipitated metal oxides.(1)

Municipal Water Treatment

Stabilization of Lime-Softened Water

The lime and lime-soda softening processes generally afford the most economical means for the reduction of calcium hardness in municipal water supplies. However, it is difficult to force the precipitation of calcium carbonate to completion with subsequent incrustation of filter beds and pipe lines through crystal growth from the super saturated calcium carbonate solution. Meticulous recarbonization can theoretically produce a calcium carbonate stable water, but in practice this is very difficult of attainment. A positive stability index must be maintained if corrosion is to be prevented, and it is a general practice to maintain such an index and yet reduce the calcium carbonate level through partial recarbonization. This reduces, but does not eliminate incrustation of filter beds and the distribution system.

The ability of minute or "threshold" amounts of sodium metaphosphate glass and certain other condensed phosphates, such as tetrasodium pyrophosphate, to stabilize supersaturated calcium carbonate solutions and prevent the precipitation of calcium carbonate has been well established. The use of sodium metaphosphate glass for the stabilization of lime and lime-soda softened waters has become common municipal waterworks practice. Feed rates of 1 to 2 ppm. have been found adequate for the stabilization of lime softening plant effluents over the pH and temperature ranges experienced in

municipal water systems with the elimination of any need for recarbonation. The resultant higher pH levels and calcium carbonate concentrations, coupled with the corrosion inhibition properties of this phosphate, have greatly facilitated corrosion control in the water system and domestic hot water heaters.(13-23)

The use of such small concentrations of sodium metaphosphate glass for this purpose obviates any consideration of its action in terms of complex formation and sequestration. Studies by Buehrer and Reitemeier(24,25) involving the precipitation of calcium carbonate from calcium bicarbonate solutions through the addition of ammonia showed that the inhibiting action of sodium metaphosphate glass is not due to its interaction with HCO_3^-, $CO_3^=$, or NH_3. These studies also disclosed that the primary precipitation reaction:

$$Ca^{++} + HCO_3^- + OH^- \longrightarrow CaCO_3 + H_2O$$

is not effected by trace amounts of sodium metaphosphate glass. Their experimental evidence regarding the inhibiting effect within the range of the threshold concentration of 1 ppm. definitely pointed to an interference in the growth of calcium carbonate crystals. It was found that the growth of the crystals was greatly retarded and the crystals formed were fewer in number, larger in size, and more or less distorted. They concluded that sodium metaphosphate glass does not interfere with the normally rapid transition of calcium carbonate crystallization through colloidal to macroscopic dimensions, but that it deranges the normal process of crystal growth through its adsorption on the crystal faces.

Corsaro, et al,(27) attempted to cast further light on the mechanism of threshold treatment of calcium carbonate systems with sodium metaphosphate glass. They obtained data which show that the threshold action appears to be dependent upon the adsorption of the phosphates from solutions of very low concentrations by the calcium carbonate nuclei as they are formed from saturated calcium carbonate solutions. The tendency to inhibit precipitation was attributed to the large ratio of double layer to particle size produced by the adsorption. This phenomenon would effect the dispersion of the ultramicroscopic crystals of calcium carbonate and prevent the regular grouping into larger crystal aggregates. When an initial concentration of sodium metaphosphate glass higher than 5 ppm. was used a tendency towards agglomeration of nuclei was indicated.

The effect of pH and temperature on the use of sodium metaphosphate glass for the stabilization of calcium carbonate must be related to time and the particular circumstances encountered. Its more rapid hydrolysis at higher pH and temperature levels is not of such magnitude as to interfere with its effectiveness under conditions of normal municipal use. Experience has shown that precipitation of calcium carbonate is prevented with threshold treatment when the pH is as high as 10. Temperatures involved in home water heaters have not presented any particular problems. Where higher temperatures are experienced, such as in boilers, supplemental conditioning would be required.

In field applications threshold treatment has shown the property of slowly removing existing calcium carbonate incrustation as well as preventing new deposition.(23) Laboratory tests have shown that the amount of sodium metaphosphate glass-treated water which must pass through an incrusted

filter before deposition ceases is directly proportional to the surface area of the incrusted filter sand and inversely proportional to the concentration of glassy phosphate.(18)

Under most circumstances dosage rates of sodium metaphosphate glass for calcium carbonate stabilization will approximate 1-2 ppm. It is common practice to start feeding 1.5 to 2 ppm. and to continue at this rate until a concentration of 0.5 ppm. is obtained at the extreme end of the distribution system. The required operating feed rate is that rate which will maintain this terminal concentration.

Following exposure to threshold treated waters calcium carbonate surfaces exhibit a hysteresis effect preventing deposition when a drop in glassy phosphate concentration below threshold level is experienced. This effect is sufficient to compensate for slight variations in feed rates.(23)

Waters high in magnesium which are treated with excess lime to the high pH necessary to precipitate the magnesium as magnesium hydroxide are not fully amendable to threshold treatment since sodium metaphosphate glass has not been found to be very effective in preventing the precipitation of magnesium hydroxide.(23,26)

Should the pH of threshold treated waters be insufficiently high to control the growth of autotrophic bacteria, chlorination to attain this control must be maintained. Otherwise, the nutrient effect of the phosphorus might stimulate the growth of undesirable quantities of these bacteria in the distribution system.

Stabilization of Iron and Manganese

Many ground water aquifers and an occasional surface water reservoir will produce waters containing varying concentrations of dissolved iron and manganese. These elements are in the reduced state when in solution, but upon exposure to dissolved oxygen or chlorine oxidize to form insoluble precipitates. To make such waters usable would normally require an iron or manganese removal plant if the combined concentration of these two elements exceeded about 0.3 ppm. However, it has been found that sodium metaphosphate glass can be successfully used in many instances to stabilize iron and manganese without need for their actual removal from the water.(16,17,28,29)

Sodium metaphosphate glass must be added before precipitation commences if satisfactory stabilization is to be obtained. This requires that it be fed prior to exposure to air or chlorine, and a point on the suction side of the well pump is usually selected for this purpose.

The successful application of this practice is generally limited to waters containing iron and manganese concentrations of less than 2.5 and 1.5 ppm. respectively. Where the iron and manganese are bound with organic matter sodium metaphosphate glass is of little or no value regardless of the concentration of these metals in solution.

The required rate of feed of sodium metaphosphate glass for iron stabilization is directly proportional to the iron content of the water. Because of this relationship, it is probable that the mechanism of stabilization is that of sequestration. A weight ratio of glassy phosphate to iron of 2:1 is generally required. Manganese is more difficult to stabilize and requires about 4 parts of glassy phosphate for each part of manganese. These feed rates are not

markedly effected by temperatures over the range 70° to 180°F or by pH conditions experienced in applicable water supplies.

This process differs from the threshold treatment used for calcium carbonate stabilization. In the latter case, the sodium metaphosphate glass concentration is relatively independent of the calcium carbonate concentration and there is no visible evidence of a disperse phase. With iron stabilization, the presence of such a phase has been clearly indicated through residue retained by filter paper.(17) Under circumstances of successful iron stabilization by glassy phosphate, the disperse phase is not of sufficient intensity to interfere with average municipal water use.

The addition of sodium metaphosphate glass to iron and mangenese bearing waters exerts a dispersing action on accumulated deposite of insoluble salts of these metals. Accordingly, frequent flushing of an old system is recommended at the start of treatment.

Chlorination adequate to control the growth of iron and manganese bacteria is a very important element to the successful prosecution of this treatment. Phosphorus is an essential bacterial nutrient and could rapidly cause growths of these autotrophic bacteria of such magnitude as to dwarf the original problem.

Well Rehabilitation

Sodium metaphosphate glass in concentrations varying from 2,000 to almost 40,000 ppm. has been successfully used for the cleaning of wells where failure was due to incrustation and plugging with mineral deposits or with clay and silt. Andrews(30) reported the use of a 2,000 ppm. sodium metaphosphate glass solution in the restoration of refinery wells. Kleber(31) recommended the addition of 16 lb. of sodium metaphosphate glass per 100 gal. of water, which is equivalent to a concentration of about 19,000 ppm. Literature distributed by a leading supplier of this compound recommends 30 lb. per 100 gal. of water, which represents a dosage rate of approximately 36,000 ppm. These greatly varying application rates reflect the many unknowns encountered in each problem and the entirely empirical nature of this type of treatment. Its effectiveness, however, has been adequately demonstrated.

The procedure used consists of adding the glassy phosphate solution together with about 5 lb. of calcium hypochlorite for every 100 lb. of phosphate to the well. These chemicals are left undisturbed in the well for 48 hours with intermittent pumping sufficient only to raise and lower water from the well to ground level. This provides the agitation necessary for dispersion and to force the solution into the surrounding strata. Following this, the well is flushed and returned to service.

Corrosion Control

Until the advent of sodium metaphosphate glass in water treatment, the method most commonly used for corrosion control in the water distribution system consisted of the formation and maintenance of a protective layer of calcium carbonate on the inner surfaces of the water pipes. This method requires critical analytical and operational control to prevent solution or excessive growth of this deposit. Its most serious limitation is its inapplicability under variable temperature conditions.

Where lime or lime-soda water softening is provided, adequate corrosion control can be attained concurrently with calcium carbonate stabilization through threshold treatment with a sodium metaphosphate glass. However, this situation does not exist in most municipal water supplies.

During the past two decades, sodium metaphosphate glass has been used with considerable success for the inhibition of corrosion in municipal and industrial water distribution systems.[32-41] The wide range of conclusions drawn from field applications and early laboratory studies can be partially attributed to the incomplete techniques frequently used for measurement of effectiveness in the field and inadequate consideration and control of all important variables in many of the laboratory experiments. Later work has emphasized the complex nature of the inhibitory action and the need for an understanding of the mechanism involved if effective corrosion inhibition is to be obtained by this means.

Eliassen and Sutherland[42] conducted laboratory tests in 1942 to determine the effectiveness of sodium metaphosphate glass for the control of corrosion of black iron pipe exposed to flowing water from the New York City supply. Runs at pH 7 and 11.5°C disclosed that 2 ppm. of the phosphate glass caused a tapering off of corrosion until the corrosion rate reached a stable value of one-fifth of the equilibrium rate with untreated water. This was substantiated by a parallel increase in dissolved oxygen with decrease in corrosion. Similar runs at pH 5 and 9 gave the same inhibition pattern.

Earlier work by Hatch and Rice[43] in 1940 disclosed a marked inhibition of corrosion of steel wool exposed to a constant flow of Pittsburg tap water containing 4 ppm. of a sodium metaphosphate glass. Oxygen absorption was used as a parameter of corrosion rate. They conclused that the inhibition of corrosion of iron and steel by sodium metaphosphate glass was probably due to the formation of an adsorbed film of the glass or some complex thereof upon the metal or metal oxide surface.

Subsequently, in 1945, Hatch and Rice[44] conducted continuous flow and batch tests using Pittsburg tap water, steel tubing and plates. Marked inhibition of corrosion in proportion to a glass concentration from 2 to 10 ppm. was reported. This work emphasized the importance of sufficient velocity of flow to provide a constant source of sodium metaphosphate glass to the metal oxide surface and the need for a minimum calcium to glass ratio of 0.2.

The mechanism of corrosion inhibition by sodium metaphosphate glass was investigated by Hatch[45] in 1952 using Pittsburg tap water and galvanic cells consisting of steel anodes and copper cathodes. He attributed the inhibitory effect of the glassy phosphate on corrosion to be the result of cathodic polarization caused by the electrodeposition of a calcium metaphosphate complex on the cathode surface. The increased polarization was found to cause a marked reduction in the current flow between anodic and cathodic members and consequently in the galvanic attack on the anodic metal. On the basis of this work, Hatch postulated that the protective glassy phosphate complex film is laid down by electrodeposition and not by adsorption as previously held. He stated that the migration of the sodium metaphosphate glass complex to the cathode in calcium bearing waters is quite different from its behavior in solutions of its sodium salts in distilled water. In the latter instance, the phosphate migrates to the anode; but in the former, it moves to the cathode as a positively charged colloidal particle or calcium glassy phosphate cation.

In a concurrent report of corrosion studies involving steel test cells and Pittsburg tap water Hatch[46] emphasized the differences between the inhibition of the corrosive attack on steel coupled to copper and the inhibition of the attack on steel alone. The first of these was the less pronounced retardation of the development of the protective film, as a result of the presence of previously formed corrosion products, in the copper-steel system than for steel alone. In this system the copper was relatively free from corrosion products after exposure to untreated water and afforded minimal obstruction to the glassy phosphate film. For steel samples exposed to an untreated water, the heavy coating of rust retarded the formation of the protective film and thus retarded rather than prevented the rapid shift of the potential of the steel in the anodic direction which is characteristic of an untreated system and indicative of the rapid breakdown of the initial oxide film. Another difference was the less pronounced hysteresis of the inhibitive action on the galvanic attack of steel coupled to copper than on the attack on steel alone.

Further work was conducted in 1952 by Mansa and Szybalski[47] who studied the influence of sodium metaphosphate glass on the potentials of iron electrodes in differential aeration celis. They reported that the glassy phosphate decreased the potential of the cathode by the formation of an adsorption layer on its surface. This resulted in the inhibition of oxygen access to the iron surface with consequent decrease of potential difference between anode and cathode, thus decreasing corrosion.

Raistrick[48] in 1952 reported that his studies indicated that a protective layer of calcium carbonate was formed on the cathodes of corrosion cells. He believed that the inhibitory action of metaphosphate could be attributed to the adsorption of the compound on the calcium carbonate layer, preventing it from passing into solution.

The most recent comprehensive investigation into the mechanism of corrosion inhibition by sodium metaphosphate glasses for which published reports are available was conducted by Lamb and Eliassen.[49,50] in 1954. Galvanic cells of steel and stainless steel were used to facilitate analyses of the action of sodium metaphosphate glass on the individual electrodes of the corrosion cell. A synthetic test water of constant chemical quality and velocity of flow was used.

The results of their studies indicated that the inhibitory action of sodium metaphosphate glass can be attributed to increased polarization of the cathode in glassy phosphate treated water. Though they found substantially greater deposits on the anodes than on the cathodes, they found that the latter were far more effective in increasing polarization and, therefore, in corrosion inhibition. Film deposition and electrical current flow studies showed that the inhibitory action of the glassy phosphate was enhanced by the presence of corrosion products in solution which increased the polarization of the cathode. The cathode film was found to contain hydrous ferric oxides, metaphosphate, calcium and possibly other chemicals. Radioactive tracer studies established that corrosion products entering solution at the anode were later deposited at the cathode of the corrosion cell. It was found that metaphosphate glass in solution reacts with corrosion products in the vicinity of the anodes, resulting in the formation of positively charged colloidal particles. These particles, containing iron oxides and metaphosphate, are subsequently deposited on the cathodes through electrodeposition. The rate of deposition of the metaphosphate glass complex on the cathode was found to be highest at

a pH of 5.0 and to decrease above and below this pH value. The film thus formed decreases the rate of corrosion of iron in water by increasing the degree of polarization of the cathodes in the corrosion cells. They concluded that, though the initial action of metaphosphate glass in short term tests may seem to be that of an anodic or mixed inhibitor, when properly used in actual pipe lines the action of metaphosphate glass would be that of a cathodic inhibitor.

The frequent use of lead service pipe has caused considerable speculation as to the effect of sodium metaphosphate glass on the solubility of lead and lead carbonate. Hatch(51) in 1941 found that 2 ppm. of glassy phosphate provided maximum inhibitive action at pH 6 with the efficacy of this treatment decreasing at pH values much below this. He found the protective action to be overshadowed by basic carbonate film formation at pH values above 7. Moore and Smith(52) conducted studies to determine whether the introduction of sodium metaphosphate glass would loosen deposits of lead salts previously formed in the pipes. In general, their conclusions were that no significant increase in pickup of lead carbonate would be expected within normal pH and metaphosphate glass concentration ranges.

Laboratory investigations and field applications have established the fact that sodium metaphosphate glass can be effective in the inhibition of corrosion of water distribution systems if properly used. The type of corrosion, however, is always the same as in the untreated water of the same pH value.(53) While recent studies have added significantly to the understanding of the mechanism involved in this process, certain factors need further investigation. The role of calcium and the effect of varying chemical quality in water have still not been clearly defined. Chemical criteria by which optimum sodium metaphosphate dosage can be determined without recourse to the trial and error technique are not available. Consequently, the application of sodium metaphosphate glass for corrosion inhibition is still fundamentally an empirical operation.

Responsible present practice in the use of sodium metaphosphate glass for the inhibition of corrosion in water distribution systems calls for an initial feed rate of about 10 ppm. Occasionally, dosages as high as 20 ppm. may be required at the onset of treatment because of high initial demand of existing corrosion products in the system.(54,55) A much higher initial and operating feed rate will be required in systems of low flow than in a system with high rates of flow since the velocity of the water influences the rate at which glassy phosphate reaches the metal and metal oxide surfaces.

Adequate control of this process can be obtained through the measurement of dissolved oxygen and dissolved iron at the point of feeding and at the extreme ends of the distribution system prior to and during the course of establishing the operating feed rate. Dosage is progressively reduced from the initial level to the optimum feed rate on the basis of the control data. This optimum rate will vary with system and water use characteristics. Corrosion control of 80-90 percent effectiveness can be obtained by this means. Because of the nutrient value of phosphates for bacterial growth, sufficient chlorination must be provided to prevent the growth of iron and other autotropic bacteria in the distribution system.

Physiological Effects

Sodium metaphosphate glass is not known to be toxic in the concentration used in water treatment. Jones(56) concluded that this compound is no more

toxic than any physiologically active salt. Gosselin, et al,[57] in experimental work with rats found the dosage required for lethal results in 50 percent of test animals to be so high as to justify classification of sodium metaphosphate glass as practically non-toxic for the single oral dose. Toxic impurities in the commercial product would seem to be of concern only when present in sufficient amount to cause an excessive concentration in the treated water. The presence of impurities at this level has not been demonstrated. No restrictions on the maximum allowable concentrations of sodium metaphosphate glass in drinking water appear in the Public Health Service Drinking Water Standards.

CONCLUSIONS

The chemistry of sodium metaphosphate glass and its present status in the treatment of municipal water supplies for calcium carbonate stabilization, iron and manganese stabilization, corrosion control, and well rehabilitation have been reviewed. This compound can be an effective tool of the sanitary engineer if applied with an informed understanding of its chemical nature, circumstances of valid application and limitations. It is hoped that this paper will contribute to that understanding.

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Paper 1465 December, 1957 SA 6

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

LAKE INTAKES*

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(Proc. Paper 1465)

INTRODUCTION

The function of any intake is to provide a dependable supply of water, adequate in quantity and of suitable quality for use and treatment. An ample supply of water is available in the Great Lakes system, but the intakes must be so located that the water quality is satisfactory and that interruptions in service will not occur because of icing or other difficulties. Sufficient hydraulic capacity must be provided for the maximum rates of water use.

Intakes comprise a lake structure, or crib, through which water enters the system and a conduit from the crib to shore facilities from which the water may be pumped.

One type of crib, exposed type, has some form of superstructure which extends above water level, from which the intake well and ports may be maintained. The Toledo, Ohio, intake is an example of this type. The submerged type of crib is entirely below the water surface. The Sandusky, Ohio, and the Plum Brook Ordnance Plant intakes are examples of the submerged type. These intakes are located in deep water remote from shore. Intake structures may also be located at or near the shore line.

This paper is in general limited to cribs and summarizes the design considerations and operating histories of several intakes.

Toledo Intake

The Toledo intake was designed in 1939 and has been in operation since 1941. It is located in Lake Erie about 2.4 miles from the shore line. The lake bottom at the crib is about at Elevation 550.7 and mean lake level is at Elevation 572.5. The intake ports are 10 feet square on the outside face, and are 16 in number. The sills of the ports are at Elevation 553.7. Thus, there is about 8.8 feet of water over the top of the port at mean lake level. At the sill elevation of the ports, the substructure wall is 19 feet in thickness. In

Note: Discussion open until May 1, 1958, Paper 1465 is part of the copyrighted Journal of the Sanitary Engineering Division of the American Society of Civil Engineers, Vol. 83, No. SA 6, December, 1957.

*Presented at a meeting of the ASCE, June 6, 1957, Buffalo, N. Y.
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passing through this wall, the ports change shape. The interior dimensions of each port are 10 feet in width by 21 feet in height.

The water passes through the ports and enters an intake well inside the structure. The intake well is 60 feet in diameter. At the center of the intake well the water enters a 9'-0" diameter conduit which conveys the water to the surge well about 2.9 miles from the crib.

The substructure extends to Elevation 582.5, or about 10 feet above mean lake level. The operating floor, at this elevation, has facilities for heating, emergency power generation, coal storage and other storage. A floor at Elevation 595.25 provides sleeping quarters and dining, flood storage, and office facilities.

The intake well is enclosed and the roof coping over this well is at Elevation 614.75, about 42 feet above mean lake level. A marine light is at the center of the structure above the roof.

The face of the substructure from Elevation 582.5 to 556.5 is faced with 3/8-inch steel plates to prevent ice damage to the concrete surfaces.

The intake was designed for a maximum capacity of 180.0 MGD. At this capacity the intake port velocity in the outside face of the crib is 0.175 f.p.s. This maximum capacity is the estimated maximum day for the year 2000.

Prior to the construction of the present intake, the City of Toledo obtained its water from the Maumee River. The principal reasons for the selection of Lake Erie for the source of water were:

1. The Maumee River at any practical point of intake was subject to sewage pollution.

2. At times the water in Maumee River in the vicinity of the intake was lowered to such an extent as to threaten failure of the supply.

3. Lake Erie water is generally softer than that of the Maumee River.

4. Lake Erie water is of more uniform quality and therefore easier to treat.

The location selected for the Lake intake showed little influence of bacterial pollution by the waters of the Maumee River. The normal hardness was 125 ppm and the maximum was about 175 ppm.

The western end of Lake Erie is rather shallow. The water depth at the intake is about 21 feet. With this depth of water, ice problems could be significant.

Frazil ice, which is apt to occur in very cold weather when the lake is free of sheet ice, may be drawn into the intake ports.

Pressure ice is found to result from wind action on sheet ice causing it to form in windrows. When this ice breaks up, large masses may float about as bergs. These bergs may damage a submerged intake and clog it, but they rarely clog an exposed intake.

Because of these ice dangers an exposed intake was selected. The ports were designed so that pressure ice could be dislodged by light charges of powder or dynamite, and the structure was designed so that frazil ice could be removed by pike poles from the inside of the structure.

The Toledo intake has been in operation about 16 years. The maximum rate of flow through the intake has been 110.0 MGD. The maximum winter rate of flow has been 75.0 MGD. At these rates, the velocities are as follows:

	Flow Through Intake	
	Winter Maximum	Maximum Rate
Port velocities fps	.07	.11

Icing in the intake ports has not been serious. At no time has the flow through the intake been stopped because of icing. From 1941 to about 1945 one operator lived at the crib, but icing has been of so little consequence that since 1945 the crib has been unattended. Pressure ice has not built up in the vicinity of the crib. At times small fish are drawn into the crib in sufficient quantities to cause quite a build-up at the traveling water screens at the shore facilities. Large fish, however, have not been troublesome.

No unusual problems of maintenance have arisen during the 16 years of operation. Maintenance requirements have been nominal.

Sandusky Intake

The Sandusky intake was designed in 1938. It is located in Lake Erie about 3,000 feet from the shore line. The intake pipe line is 42 inches in diameter, and crosses Sandusky Bay. The total length from the submerged crib to the suction well is about 7,800 feet.

The crib is a nine-celled timber structure, of white oak timbers, 30 feet by 30 feet in plan and 15.5 feet deep. The bottom of the timber crib is keyed into 10 feet of clay overlaying the solid rock, and the top of the crib extends 5 feet above the normal lake bottom. The clay lake bottom adjacent to the crib was excavated to solid rock and backfilled with large rock approximating 0.5 to 2.0 tons each in weight. This rock paved area adjacent to the crib structure is laid on a 2-1/2 to 1 slope. The water depth over the crib is 19 feet. The water depth in the lake adjacent to the crib is 24 feet, referred to mean lake level.

The central cell of the nine-celled crib structure is 20 x 20 feet in plan and houses the intake drum. The eight outside cells of the structure are filled with large rock.

The intake drum consists of a vertical steel inverted truncated cone 8 feet in diameter at the base, 14 feet in diameter at the top and 4 feet high. Over the top rim of this shaft is a flat steel roof or cover. The drum is mounted on a vertical shaft 8 feet in diameter. The inlet port area is protected by wooden racks or slats bolted to the steel structure. All steel surfaces of this structure are coated with a hot application of coal tar pitch.

At the shore line of Sandusky Bay an emergency intake has been provided. This emergency intake is a valved connection into the intake pipe through which water from a rock filled timber crib may be drawn.

Prior to the construction of the present intake, Sandusky secured its water from an intake in Sandusky Bay. A lake location for a new intake was considered because of the increasing sewage and industrial waste pollution in Sandusky Bay and because at times the water supply was threatened by storms which lowered the water level in the Bay below the intake ports. From chemical and sanitary analyses of the water, it was concluded that a satisfactory water supply could be obtained at the site of the proposed Lake intake.

The icing problems in the Sandusky area are similar to those at Toledo. The proposed intake was to have a capacity of 20.0 MGD. For this capacity the use of an exposed intake was considered uneconomical. To prevent, insofar as practical, the introduction of frazil ice, the intake ports were designed so that the entrance velocity would not exceed 0.25 foot per second. The timber crib with rock riprap and rock cells was designed so that the large masses of floating ice would not do structural damage to the intake crib.

This lake intake has now been in operation about 17 years. The maximum flow through the intake has been 32.0 MGD and the maximum winter flow through the intake has been 28.0 MGD. The maximum port velocity for winter operation thus has been 0.35 fps.

Neither frazil ice nor pressure ice has been troublesome at Sandusky. The raw water has been satisfactory both in the chemical and sanitary quality. Fish have not been troublesome, and no problem of maintenance has arisen.

Plum Brook Ordnance Plant

Late in 1941 an intake was constructed for the Plum Brook Ordnance Works. This intake is of the submerged type and is located in Lake Erie about 1.5 miles southeast of the Sandusky intake. It is quite similar to the Sandusky intake. A rock-filled, nine-celled timber crib was used, but because of war-time shortages, the steel intake drum was not used. Instead, a horizontal timber flare piece on the end of a 42-inch steel pipe was provided in the center well. The center well was covered with timbers and water entered the center well through horizontal slots above the stone dyke.

Considerable trouble was experienced with this intake. Ice does not appear to have been a problem, but during storms large amounts of sand, clam shells, sticks, roots, and other material entered the intake. At times the quantity of sand restricted the supply.

The reasons why the sand and debris entered the Plum Brook intake were never clearly established. The top of the stone dykes around the crib was about 5 feet above the lake bottom, but the borings indicate that some sand was originally present at the location of the crib. The currents during storms may have been sufficient to suspend the sand, and, with it, the other debris. The horizontal flare piece, near the bottom of the center well may have fostered the introduction of sand and other foreign material into the intake piping.

The difficulty with sand, assuming that it entered the pipe at the crib and not through a break in the pipe line, emphasizes the importance of location on satisfactory performance.

Shore Intakes

Deep water intakes provide a means of securing dependable supplies of good quality, but intakes at or near the shore may provide an economical means of supplementing deep water supplies. The considerations of raw water quality have resulted in the selection of locations of intake remote from the shore. For example, until recently, the only treatment provided for any Chicago water was sterilization, and at the present time this is the only treatment provided for the water supplied to about two-thirds of the

City. With treatment limited to sterilization, the quality of the raw water is of major importance. When Chicago constructed the South District Water Filtration Plant, a shore intake was provided to supplement the crib intake. The Central District Water Filtration Plant, now under construction, will also include a shore intake to supplement the crib supply. These shore intakes would not be practical without the facilities for coagulation, sedimentation and filtration included in these Filtration Plants.

At Highland Park, Illinois, on Lake Michigan, the deep water intakes provided relatively ice free operation, but the capacity of the intakes was insufficient to meet summer peaks. A valved inlet was installed on a 20-inch intake pipe line to supplement the supply. This inlet is located about 800 feet from shore in about 12 feet of water. The supplementary inlet is beyond the "sand line." The valve is operated from shore and may be closed whenever the quality of the raw water proves unsatisfactory. This supplementary inlet has been in operation less than one year. The operating history, therefore, is too short to draw any final conclusions. The inlet, however, appears to be satisfactory for the purposes for which it was installed.

For supplementary intakes near shore in relatively shallow water, mechanical protection against damage by anchor ice is of great importance. If there is any danger of sand piling up around the inlet, protection against the introduction of sand into the intake is also necessary. Whether or not supplementary intakes, such as was provided at Highland Park, will be satisfactory for year-round operation is not yet known. The Highland Park supplementary intake was operated during the winter without difficulty, but several years of operation will be necessary before winter operating conditions may be appraised. It is thought that since frazil ice does not appear to form when the lake surface is covered with sheet ice, the "shore" intake may provide relatively ice-free operation during severe winters.

CONCLUSIONS

The operating histories of the Toledo and Sandusky intakes indicate the following:

a) With low intake velocities and protection against pressure ice, serious icing problems can be avoided in both exposed and submerged intakes.

b) The location of the intake is important in securing a water of good sanitary, physical, and chemical quality.

The experience with the Plum Brook Ordnance Works intake may emphasize the importance of location. It has not been possible to obtain full data on the source of sand, but the original presence of sand at the crib site suggests that sand may be suspended and drawn into the intake during storms.

Both submerged and exposed intakes can be constructed in the Great Lakes to provide a dependable supply of water of satisfactory quality. The intakes should be located in the greatest practicable depth of water at sites from which a good quality of water may be obtained and should be designed so as to produce low velocities through the inlet ports.

Shore intakes may be practical and economical to supplement crib supplies if the physical, chemical, and sanitary quality of the water near the shore is satisfactory. It is recognized that the diversion of Chicago's sewage from

Lake Michigan contributes to the acceptable sanitary quality of the Lake water near the shore in the Chicago area, but improvements in the methods of water treatment, and the lessening of pollution because of increased treatment of sewage and industrial wastes, may make supplementary shore intakes safe, practical and economical.

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

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* There will be no closure.

FLOW OF CONCENTRATED RAW SEWERAGE SLUDGES IN PIPES[a]

Discussion by R. N. Roberts

R. N. ROBERTS.[1]—Mr. Brisbin's straightforward approach is commendable. His use of full-scale equipment obviates the usual problem of extrapolating test data obtained from models. Since the purpose of this investigation was to secure information for the design of sludge lines when applying reciprocating pumps, the procedure was practical and the results are useful.

Although it is not common practice to apply centrifugal pumps for this service, they have been so used. As an employee of a centrifugal pump manufacturer, the writer is continually involved in the proper selection of design values for pipe lines handling non-Newtonian fluids very similar to the one under discussion. With these pumps, the system head determines the capacity. Thus useing data as presented in Figure C and D would not be sufficiently accurate for such applications.

This difficulty can easily be overcome by re-arranging the test results presented in Mr. Brisbin's paper. Several investigators, including Babbitt and Caldwell, suggest the plotting of shearing stress versus inverse seconds to obtain a graph usable with all sizes of pipe. Taking the four points available for each consistency as given by the curves in Figures C and D and following this suggestion would result in a family of curves as shown by the accompanying chart. (Fig. 1)

Employing this treatment of the test data would reduce the amount of interpolation required when designing sludge lines.

Granted the four points do not clearly define the shape of the curve. However, since sewerage sludges have been proven to be non-Newtonian, it is reasonable to assume the curve between the values of $V/4D = 1$ and $V/4D = 4$ is a straight line.

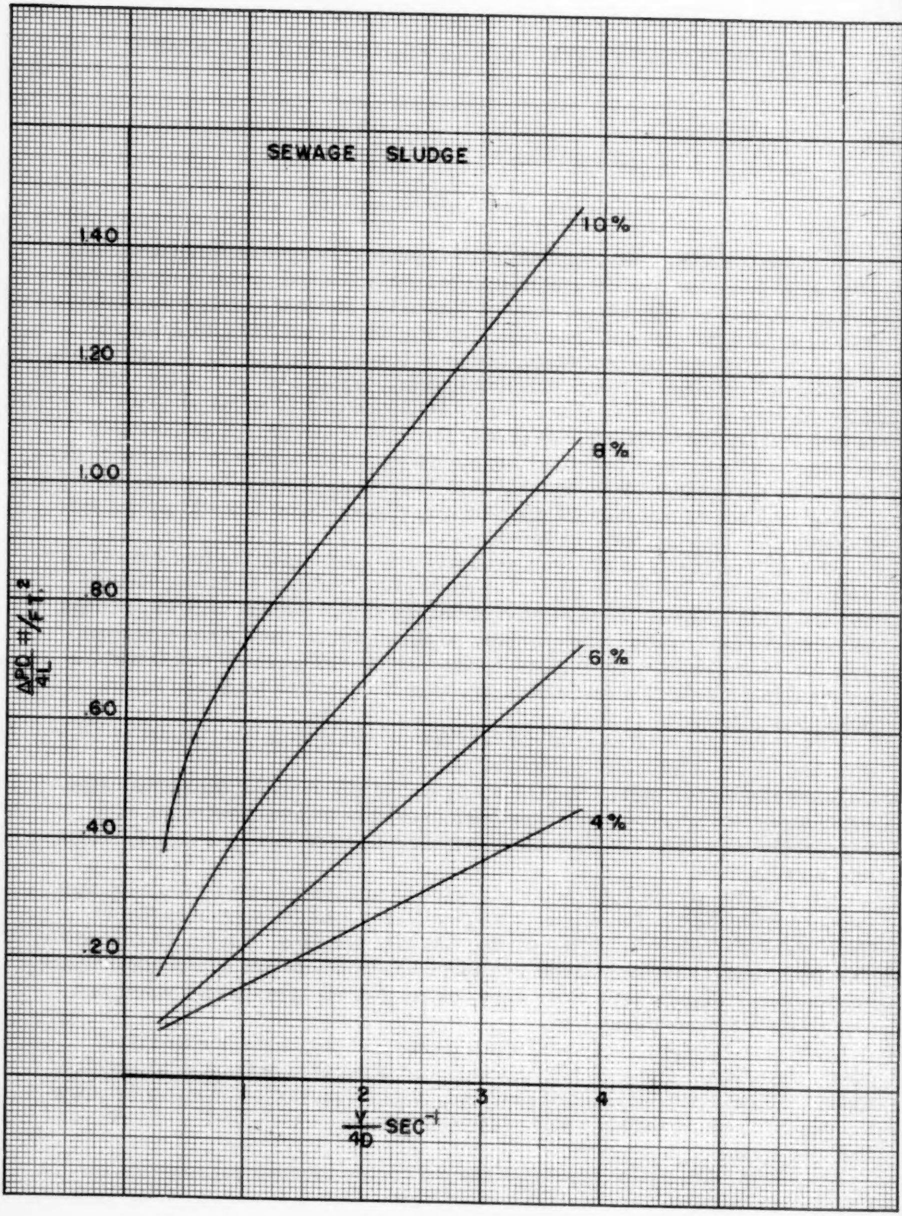
The validity of this approach is attested to in a small way by the readings taken at Greenwich. The shear rates (V/4D) for 1.53 ft./second in the 4" lines and 2.25 ft./second in the 6" line are 1.15 seconds $^{-1}$ and 1.13 seconds $^{-1}$ respectively. The unit shear values for these points are within 10% at each consistency. This is very close considering that the investigator was handicapped by the intermittent flow.

Care should be taken to avoid using points that would involve turbulent flow. Reynold's critical can be computed by using the apparent viscosity for the velocity and pipe diameters selected. Should the desired capacities place the design in the turbulent range, different procedure for friction determination is required.

In table B, Mr. Brisbin presents the seemingly paradoxical fact that in

a. Proc. Paper 1274, June, 1957, by Sterling G. Brisbon.

1. Test Engr., Morris Machine Works, Baldwinsville, N. Y.


SEWAGE SLUDGE
10%
8%
6%
4%
1.40
1.20
1.00
.80
.60
.40
.20
2
3
4
$\frac{\Delta PD}{4L}$ #/FT.^{2}
$\frac{V}{4D}$ SEC^{-1}

some instances involving elbows, an increase in velocity and/or percent solids will decrease the head loss due to friction. An explanation of this is apparent when one recalls that in the laminar range, resistance to flow is not greatly affected by the roughness of the pipe walls, or deviation of the path from a straight line. Undoubtedly at 0.9% solids, and 5.18 ft. per second, the flow was turbulent, but increasing the solids content to 9.1% changed the character of the fluid causing laminar flow. This is a small point in most sewage projects, but should a pipe line include many fittings, the risk of over design is encountered if an average obtained from so great a spread is used.

Pipe line and pump engineers should feel indebted to Mr. Brisbin for his significant addition of useful data to a field of study that needs more serious attention.

DIRECT RECHARGE OF GROUND WATER WITH SEWAGE EFFLUENTS[a]

Discussion by Ralph Stone

RALPH STONE,* M. ASCE.—The authors have contributed valuable information to the art of injection well ground water recharge employing sewage effluents. Further experience on this subject is herewith presented in order to amplify and compare The University of California, Richmond, tests with that obtained at the existing full scale Mattoon, Illinois operation and the Los Angeles County Flood Control District injection well test currently taking place at the Hyperion Sewage Treatment Plant of the City of Los Angeles, California. It is not intended to present at this time the detailed results of these injection well operations.

At Mattoon, Illinois, The Carter Oil Company reports[1] that in 1954 it contracted for the purchase of 0.21 mgd of municipal sewage effluent. This secondary treated effluent is further polished by means of sand filtration and chlorination. Since 1954, the reclaimed water has been injected continuously into recharge wells. These recharge wells have never required further redevelopment. The wells are operated satisfactorily at relatively high pressures applicable for injecting the reclaimed water into the deep oil stratums. At other oil fields it has been found suitable to inject waste oil field brines back into the oil stratum in order to obtain secondary recovery of additional oil and gas.

At the Hyperion Test Site[2] it has been found unnecessary to use high chlorination dosages either for well redevelopment or for reclaimed water injection. In fact, no chlorination has been employed in redeveloping the injection test well by means of bailing and surging with a conventional cable tool well drilling rig. Continuous dosage concentrations (10 - 20 ppm) caused corrosion products to form which appear to have quickly reduced the recharge well permeability.

The Los Angeles County Flood Control District has been able to reclaim by means of intermittent sand filtration the high rate activated sludge Hyperion effluent. Following a dosage with 0.5 - 2.0 ppm total chlorine, this reclaimed water was injected into a test well continuously for a period of over six months at the rate of 0.3 cfs. and provided only minor head build-up in the receiving well. The injection well operating characteristics appear to have been similar during this well run to that resulting with the use of the imported domestic supply.

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a. Proc. Paper 1335, August, 1957, by R. B. Krone, P. H. McGauhey and H. B. Gotaas.

* Cons. Engr., Ralph Stone and Co., Engrs., Beverly Hills, Calif.

1. Private communication.
2. Los Angeles County Flood Control District information.

This Hyperion test experience has demonstrated that low-cost, reclaimed-water, well-injection recharge necessitates the use of a highly stabilized reclaimed water containing low BOD and suspended solids.

The chlorine dosage (0.5 - 2.0 ppm) at the Hyperion Test has successfully controlled coliform organisms and the injected water has consistently met recommended bacterial standards promulgated by the USPHS.

It is of further interest to note that the reclaimed water has been routinely analyzed for deleterious minerals and has proven to be within the recommended mineral quality requirements for irrigation and potable waters. Even detergents have been reduced in concentration from 12 ppm as (ABS) Alkyl Benzene Sulphonate in the crude Hyperion sewage to about 1 ppm in the reclaimed test injection well water.

SEWAGE TREATMENT BY RAW SEWAGE STABILIZATION PONDS[a]

Discussion by Ralph Stone

RALPH STONE,* M. ASCE.—Messrs. Towne and Davis are to be commended for the clarity and detail of their excellent paper concerning waste stabilization ponds. The writer has been privileged in the last three years to have visited and observed many operating Texas and other western lagoons and to have been associated with the design and operation of the Mojave Waste Stabilization Ponds experiments for over two years in conjunction with the University of Southern California "Waste Water Reclamation Project" headed by Professor Robert C. Merz and supported by the California State Water Pollution Control Board. In addition the writer has been the design engineer on several new raw sewage stabilization ponds installations.

The further comments herewith presented are intended to expand the engineering value of the experience developed by Towne and Davis.

Careful hydraulic studies at various locations employing flourescent dyes and other flow techniques have demonstrated clearly that shallow basins of less than 3 to 4 ft. depths appear to provide a poor mixing environment. In these shallow basins of less than 3 to 4 ft. depths, it was common to observe the by-passing of crude sewage through the primary basins, as well as the odoriferous floatation of the grey relatively warm crude sewage on the colder basin waters during winter and fall seasons. In contrast, deeper basins of 5 to 8 ft. depths normally enabled the masking of the inflowing sewage. With a wind velocity of about 10 mph excellent mixing occured. It is, no doubt, simple to maintain aerobic conditions in a lightly loaded basin, either shallow or deep, provided the average BOD loading is less than 25 pounds BOD per acre. However, utilizing a deeper basin it was found possible to continuously operate, at the Mojave Test installation (8 ft. deep) during summer and fall months (June-Sept. 1957) at an average loading of over 300 pounds BOD per acre accompanied by slight odors. This basin was green colored from dominant chlamydamonas algae present during this operating period. Chlamydamonas was also present when operating the test basin with lesser loadings (1955-1957).

The winter operation of a raw sewage stabilization pond may result in the accumulation of organic sludge banks which in the warmer spring temperatures may be responsible for anaerobic odor nuisances. A possible method for improving stabilization pond operations would be to employ additional treatment facilities such as a trickling filter or additional basin surface areas to alleviate the spring time or other temporary high loadings. Experience indicates that water can be conserved and storage losses reduced

a. Proc. Paper 1337, August, 1957, by W. W. Towne and W. H. Davis.

* Cons. Engr., Ralph Stone and Co., Engrs., Beverly Hills, Calif.

by increasing the loading rate into the basins during warm weather. Further observations of established lagoons demonstrated that the basin percolation rate is reduced from an initial rate of as much as several feet in one day to as little as a 1/4 of an inch per day after several years of continuous spreading. (It is also possible to percolate large quantities of effluent continuously by use of optimum waste water spreading techniques.)

Secondary ponds following primary crude sewage basins serve the following valuable functions: (1) Entrapment and prevention of by-passing objectionable solids or unstabilized crude sewage; and (2) Provide a wet well reservoir source for a pumping plant; or (3) Enable the evaporation and percolation or excess waters. The secondary sewage ponds require only 1 day detention time in volume and need not be designed for BOD loading if the primary basin is aerobic. Experience has demonstrated that it is generally unnecessary to employ: recirculation via mechanical pumping, comminutors or other special devices. Experienced engineering practice can foresee operational and construction difficulties and enable a most simple and efficient treatment system.

A shallow aerobic basin has the advantage of maintaining the growth of certain aerobic algae whereas the deeper basins favor Chlamydamonas and other "facultative" type algae that can exist when little dissolved oxygen is present. An optimum stabilization pond design possibly would employ basic technical and construction factors so as to provide good mixing by introducing the crude sewage into a relatively deep (5 - 8 ft.) center section of a large basin while providing for a shallow (3 ft. deep) perimeter buffer area. Such a basin would have the added advantages of requiring minimum earth handling and dike construction as well as assisting in the initial start-up operation of a lagoon system within the center sub-basin area.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

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c. Discussion of several papers, grouped by Divisions.

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